LOAD CARRYING CAPACITY OF REINFORCED POND ASH BEDS

A Thesis submitted in partial fulfillment of the requirements for the award of the degree

Master of Technology

In

Civil Engineering

(Geotechnical Engineering)

By

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DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY
ROURKELA-769008, INDIA

MAY- 2014

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CERTIFICATE

This to certify that the thesis entitled **"LOAD CARRYING CAPACITY OF REINFORCED POND ASH BEDS"** being submitted by **SRIKANTH BANDELA** in the partial fulfillment of the requirements for the award of Master of Technology Degree in **Civil Engineering** with specialization in **GEOTECHNICAL ENGINEERING** at the National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other university/institute for the award of any degree or diploma.

Prof. Suresh Prasad Singh

Department of Civil Engineering

National Institute of Technology

Rourkela – 769008

Date:

DEDICATED TO

My Parents

ABSTRACT:

There is required for increasing the usage of coal ashes. Pond ash being a non-plastic cohesion less material has potential to be used as a fill material in Geotechnical structures like retaining walls, embankment, structural land filling etc. However the literature review shows compacted pond ash, which gives similar strain that of the similar graded earth material in dry / partially wet condition loses its shear strength substantially upon the saturation. So its most of the design recommendations provisions are made to keep the water away from the compacted pond ash fills by providing a layer of compacted soil around it. However the strength of the compacted pond ash fills can be retained partially by reinforcing it suitably. Soil reinforcement technique is one in all the more standard techniques used for improvement of poor soils. Metal strips, synthetic geotextiles, geogrid sheets, natural geotextiles, at random distributed, synthetic and natural fibers area unit getting used as reinforcing materials to soil. Further, the soil reinforcement causes important improvement in physical property, shear strength, different properties, bearing capacity and economy. This can be a comparatively easy technique for ground improvement and has tremendous potential as a cost effective answer to several Geotechnical issues. Keeping this in mine associate experimental study is taken up to improve the strength of compacted pond ash. This has been achieved in two series of tests. In the first series of test the compacted pond ash beds are reinforced with either PVC net or GI nets. The size of the reinforcement as well as the position of the reinforcement was varied. The CBR value of these specimens we are found out and these are compared with that of and unreinforced pond ash bed compacted to MDD at OMC. In the second series of the tests the CBR value of river sand compacted to different relative densities and different moisture contents are evaluated. In general the CBR value of compacted sand at different moisture content are found to be higher than compacted pond ash. To find out the CBR values of compacted pond ash beds overlain by a layer of sand with different relative densities and moisture contents. The optimum thickness of sand beds has been found out such that effect of underlain pond ash beds is minimal on the CBR value based on these experimental results it is concluded that in filed the bearing capacity of ash beds can be improve either by reinforcing it with suitable gird reinforcing materials are by overlaying it with suitable thickness of stiffer material such that the effect of under laid Pond ash material is not felt.

ACKNOWLEDGEMENT

The satisfaction on the successful completion of any task would be incomplete without the mention of the people who made it possible whose constant guidance and encouragement crowned out effort with success.

I am grateful to the **Dept. of Civil Engineering, NIT ROURKELA,** for giving me the opportunity to execute this project, which is an integral part of the curriculum in M. Tech program me at the National Institute of Technology, Rourkela.

I would like to take this opportunity to express heartfelt gratitude for my project guide **Dr. S. P. Singh,** who provided me with valuable inputs at the critical stages of this project execution. I would also like to express my gratitude to Prof. N. Roy, Head Civil Engineering Department, Prof. C. R. Patra and Prof. S. K. Das for their help and constructive suggestions during the project work. My special thanks to Civil Engineering Department, for all the facilities provided to successfully complete this work. I am also very thankful to all the faculty members of the department, especially Geo-Technical Engineering specialization for their constant encouragement during the project. I am also thankful to staff members of soil engineering laboratory especially Mr. Chamuru Suniani and Mr. Narayan Mohanty for their assistance and co-operation during the course of experimentation.

The help and support received from my friends Ganesh Ravi, Preety Nanda Nanda, Subhasree samanta singhar and many more who made constructive comments and helped physically during the project work.

Finally, I would like to thank my parents and especially my brother who continuously motivated me.

Srikanth Bandela

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M. Tech Geo Technical Engineering

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CHAPTER -1

INTRODUCTION

1. INTRODUCTION:

There is required for increasing the usage of coal ashes. Pond Ash being a non-plastic cohesion less material has potential to be used as a fill material in Geotechnical structures like retaining wall, embankment etc.,. Soil reinforcement technique is one in all the more standard techniques used for improvement of poor soils. Metal strips, synthetic Geotextiles, geogrid sheets, natural Geotextiles, at random distributed, synthetic and natural fibers are getting a unit used as reinforcing materials to the soil. Further, the soil reinforcement causes important improvement in physical property, shear strength, different properties, bearing capacity and economy. This can be a comparatively easy technique for ground improvement and has tremendous potential as a cost effective answer to several Geotechnical issues. Keeping this insight associate experimental study is going to be done using geotextile and sand layer. The Geotextile layer area unit organized at intervals the soil sample with varied soil layer thickness and density of sand and pond ash. A laboratory model test for bearing capacity determination will be done similar to every combination of reinforcement with varied geotextile depth ratio and varying thickness of the sand layer. Further, these test results will be compared there upon of unreinforced soil.

The current electricity generation in India is regarded concerning, 12,058 MW, 65- 70th of that is thermal (mostly coal based). According to an estimate 100,000 Capacity or additional would be needed within the next ten years as a result of regularly increasing demand for electricity. In India ash generation is around 170 million tons / year and is about to continue at a high rate into the predictable future. Fly ash is that the residue of the coal combustion method in power plants. Nearly 73 of India’s total installed power generation capacity is thermal, of that coal primarily based generation is almost 90 % (diesel, wind, gas & steam adding to concerning 10 percent). The 85 utility thermal power stations, additionally too many captive power plants, use bituminous or sub-bituminous coal and manufacture large volumes of fly ash. High ash content (30-40%) of Indian coals is contributing to those large volumes of fly ash. At present, nearly a hundred and seventy million tons of fly ash are being generated annually in India and nearly 65,000 acres of land are presently occupied by ash ponds. India’s dependence on coal as a source of energy shall continue within the next millennium and so fly ash management would remain a very important area of national concern. Its indiscriminate disposal needs large volumes of land, water and energy. Pond ash deposit posses’ high compressibility, low bearing capacity, therefore acres of land gets wasted. Fly ash can be stabilized using a compacted stone column to increase the bearing capacity and structures may be engineered on ash pond during a price effective manner.

CHAPTER 2

LITERATURE REVIEW

2. LITERATURE REVIEW:

2.1 POND ASH

An ash pond is a designed structure for the disposal of bottom ash and fly ash. The wet disposal of ash into ash ponds is that the most common ash disposal methodology helpful in different ways includes dry disposal in landfills. Dry-handled ash is commonly recycled into helpful building materials. Wet disposal has been more popular because of economic reasons, but increasing environmental relating to concerning locate from ponds has methodology the recognition of wet disposal. The wet methodology consists of constructing a large pond and filling it with ash slurry, permitting the water to drain and evaporate from the ash over time. The ash pond area unit typically shaped using a ring embankment to surround the disposal site. The embankments square measure styled using similar design parameters as embankment dams, together with zoned construction with clay cores. The planning method is primarily centered on handling seepage and making certain slope stability.

Reinforced earth is a material that could be a combination of soil and reinforcement, appropriately placed to withstand the developed tensile stresses and additionally it improves the resistance of the soil within the direction of the greatest stress. The essential options of reinforced earth are the friction between the earth and reinforcement, by means of friction the soil transfer to the reinforcement the forces built on the earth mass. The reinforcement has therefore developed tension when the earth mass is subjected to shear stresses along the reinforcement.

2.1.1 SOURCE OF MATERIALS:

Pond Ash: The pond ash was collected in gunny bags from NTPC Kanhia, Odisha. It was dried in the oven at 105˚C-110˚C and kept in an airtight container for further use.

Sand: In the present study the sand is collected from a nearby Koil river bed, Rourkela, Odisha.

Reinforcing Materials: PVC net having 1mm aperture was used for encasing the SC. A GI strip of 1mm opening and PVC net were used as circular parallel strips for reinforcing action. Tensile strength test was carried out on the materials and the values obtained are presented in

2.1.2 Uses of Pond Ash:

In Land fill and dyke raising.

In Structural fill for reclaiming low areas.

Manufacture of Portland cement.

Lime – fly ash Soil stabilizing in Pavement and Sub-base.

In Soil conditioning.

Manufacture of Bricks.

Part replacement in mortar and concrete.

Stowing materials for mines.

2.1.3 LIST OF INDUSTRIES GENERATING POND ASH/FLY ASH

Table 2.1 List of Industries Generating Pond Ash/Fly Ash

Name Of The Industry	Name Of The State Situated	Name Of The Place
Kothagudem	Andhra Pradesh	Nellore
Ramagundam	Andhra Pradesh	Vijaywada
Bongaigaon	Assam	Lakwa
Narup	Jharkhand	Chandrapura
Barauni	Jharkhand	Bokaro
Chandradurg	Bihar	Muzzafarpur
Patratu	Jharkhand	Ramgarh
Indraprasta	Delhi	Rajghat

Badarpur	Delhi	Mathura Road
Utraw	Gujarat	Gandhinagar
Sabarmati		Utkai
Wanakoi		
Singrauli	Uttar Pradesh	Mirjapur
Rihand		Panki
Paricha		Anapara
Obra		Rpc
Hardoganj		Tanda
Ferojgandhi		
Korba	Madhya Pradesh	Satpura
Amarkantak	Madhya Pradesh	Vindhyachal
Gurunanak Dev	Bathinda	Ropar
Kota		
Raichur	Karnataka	
Ennore	Tamilnadu	Tuticorin

Mettur		Neyveli
Trombay	Maharastra	Nasik
Chola		Bhusawal
Chandanpur		Koradi
Talcher	Orissa	
Durgapur	West Bengal	Bundel
Santadir		Lolaghat
Farakka		DPL
C.E.S.C		Titalagarh
New Cossipore		Mulajore

(B) Steel Industry

Name Of The Industry	**Name Of The State Situated**
Bhillai Steel	Madhya Pradesh
Durgapur Steel	West Bengal
Rourkela Steel	Odisha

Bokaro Steel	Jharkhand
HSCO	Burnapur,(W.B)
Salem Steel	Tamil Nadu
Visakhapatnam Steel	Andhra Pradesh

(C) Aluminium Industry

Name Of The Industry	**Name Of The State Situated**
BALCO	Korba, (M.P)
NALCO	Odisha

(D) Copper Industry

Name Of The Industry	**Name Of The State Situated**
Chandmari Copper Project	Rajasthan
Khetri Copper Project	Rajasthan
Dariba Copper Project	Rajasthan
Indian Copper Complex	Bihar
Rakha Copper Project	Bihar
Malanjkhand Copper Project	Madhya Pradesh

2.1.4 FACTORS AFFECTING PROPERTIES OF POND ASH:

Effective utilization of pond ash in Geo-technical constructions as a replacement to standard earth materials needs special attention. The inherent strength of the compacted pond ash mass reduces significantly because of saturation. During this context to enhance and retain the strength of compacted pond ash, cementing agents like cement or lime could also be substantially useful. The stress-strain behavior of compacted pond ash mass is changed by inclusion of fiber reinforcements. Fiber reinforcements additionally improve the strength characteristics of the mass. Although, the utilization of reinforced earth materials has been widely accepted in several areas like embankments, foundations medium, railroads, retentive walls, however the use of pond ash in situ of earth material has not drawn much attention of researchers.

. 2.2 USE OF REINFORCEMENT FOR IMPROVEMENT IN BEARING CAPACITY:

2.2.1 GENERAL MODES OF SHEAR FALURE:

Experimental investigations have indicated that the foundations on dense sand with relative density larger than 70 % fail suddenly with pronounced peak resistance once the settlement reaches concerning 7 % of the foundation breadth. The failure is among the looks of failure surfaces and by considerable bulging of a sheared mass of sand. This kind of failure is designated as a general shear failure by Terzaghi (1943). Foundations on sand of relative density lying between 35 and 70 % don't show a sudden failure because the settlement exceeds regarding 8 % of the foundation breadth, bulging of sand starts at the surface. At settlements of concerning 15 % of foundation breadth, a clear boundary of sheared zones on the surface seems. However, the peak of base resistance may never be reached. This type of failure is termed a local shear failure.

Vesic (1963) The three types of failure represented higher than during tests on model footings. It’s going to be noted here that because the relative depth/width ratio will increase, the limiting relative densities at which failure types change increase. The approximate limits of the types of failure to be affected as relative depth DJB, and relative density of sand, Dr. vary are shown in Fig. 2.1 an equivalent figure shows that there's an essential relative depth below that only punching shear failure occurs. For circular foundations, this essential relative depth, DJB, is around 4 and for long rectangular foundations around 8.

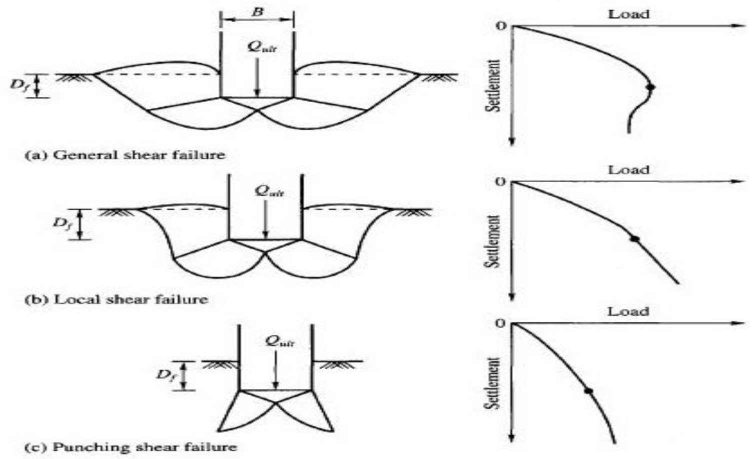


Fig 2.1 Modes of bearing capacity failure (vesic, 1963)

2.2.2 BEARING CAPACITY IMPROVEMENT OF SOIL USING REINFORCEMENTS:

H.P.Singh et al., [2012] Experimental study was conducted with regionally obtainable (Itanagar, Arunachal Pradesh, India) soil reinforcement with jute geotextile layers. The Jute Geotextile layers are organized inside the soil sample in several combinations like 1 layer, 2 layers, 3 layers, 4 layers etc. And laboratory CBR values were determined in each soaked and unsoaked condition comparable to each combination of reinforcing layer.Further, these test results were compared therewith of unreinforced soil. It had been determined that inclusion of Jute Geotextile layer will increase the CBR value of soil and this increase is more similar to 4 layers of Jute Geotextile layers. Thus, there's a significant increase in CBR value of soil as a result of inclusion of Jute Geotextile layers as a reinforcement.

Binquet, et al. [1975] were the first to report a systematic study on bearing capacity of reinforced soil beds. Their study included model plate load tests with parametric variation and proposing a method of analysis and design. Then conducted model tests on 76.2mm wide strip footing on sand reinforced with aluminum strips. The tests were conducted for the following three conditions.

1. Deep homogeneous sand layer

2. Sand layer over an extensive layer of very soft material simulating soft clay or peat

3. Sand layer above a finite sized pocket of a very soft material such as a pocket of organic soiloracaver in lime stone.

In all three series of tests, they have investigated the effect of number of layers and the depth of the topmost layer of the reinforcement on the bearing capacity. A new term bearing capacity ratio has been defined to compare the test data as:

$$\text{BCR} = {}^{q}/_{q_o}$$

Where

qo is the average contact pressure of the footing on the unreinforced soil for a given settlement.

q is the average contact pressure of the footing on reinforced soil bed at the same settlement.

The following three modes are failures have been proposed for the reinforcement soil bed.

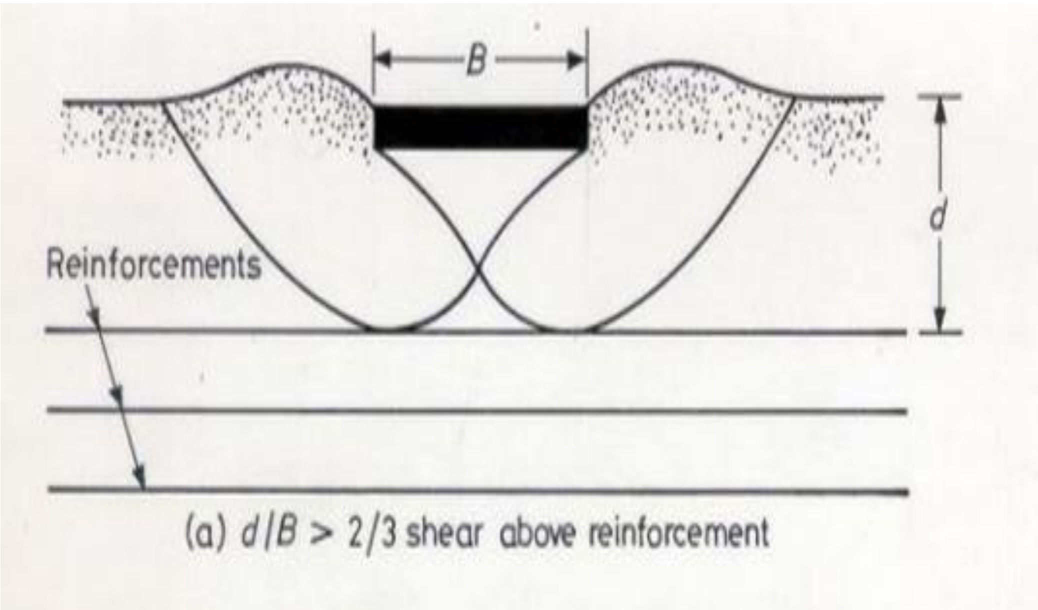


Fig (a) modes of failure in upper most layer of the reinforced

Shear failure of soil above the uppermost layer of the reinforcement –this mode of failure is possible if the depth of the topmost layer of reinforcement is sufficiently large so as to form an effective boundary into which the shear zones cannot penetrate.

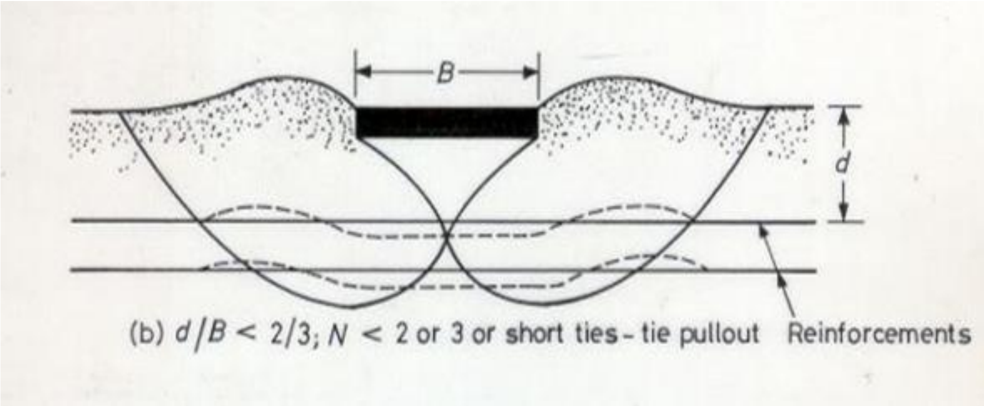


Fig (b) modes of reinforcement pull out failure

Reinforcement pull out failure – this type of failure occurs for reinforcement placed at shallow depths beneath the footing and /or reinforcements which have insufficient anchorage.

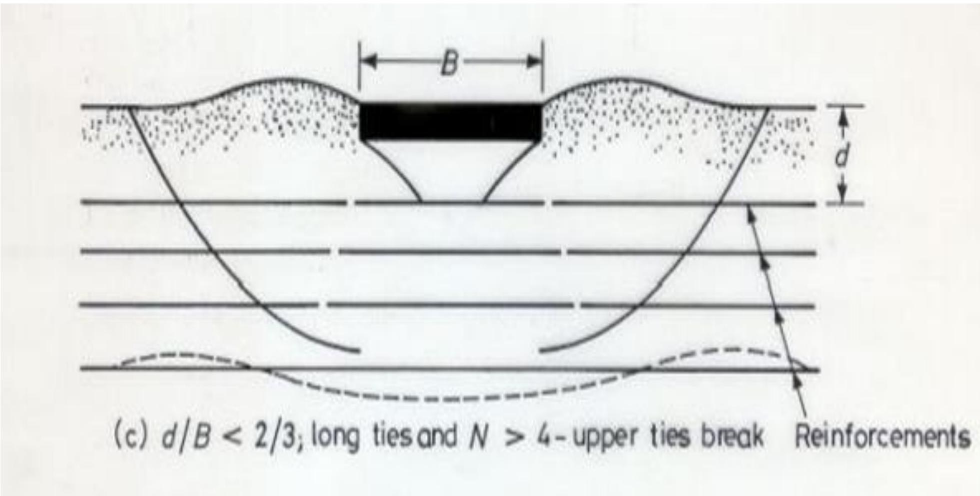


Fig (c) modes of tension failure

Reinforcement tension failure-this type of failure occurs in the case of long and shallow reinforcements for which the functional pull outs resistance is more than the tensile strength.

Binquet and Lee have thought-about, whereas proposing an analytical methodology for the planning of reinforced soil beds ,only reinforcement pull out and reinforcement tension failures .Assuring boussinesq's stress distribution to a lower place the footing to the valid ,location of maximum tensile stress within the reinforcement has been defined .It has been assumed that for associate applied a load , the tensile force within the reinforcement varies reciprocally with the total volume of the reinforcement

Using sheet reinforcement:

Wasti Y., Butun M.D., [1996]. A series of laboratory model tests on a strip footing supported by sand reinforced by at random distributed polypropylene fiber and mesh components was conducted in order to match the results with those obtained from a reinforced sand and with each other. For conducting the model tests, uniform sand was compacted within the test box at its optimum moisture content and maximum dry density. 3 varieties of reinforcement, 2 sizes of mesh elements having an equivalent opening size and one size of fiber element cut from the meshes, were utilized in varied amounts in the tests. Results indicated that reinforcement of sand by at random distributed inclusions caused an increase in the ultimate bearing capacity values and also the settlement at the ultimate load in general. The effectiveness of separate reinforcing elements was determined to depend on the quantity as well as the shape of the inclusions. The larger mesh size was found to be superior to different inclusions considering the ultimate bearing capacity values. For the mesh elements there seems to be an optimum in collusion ratio, whereas fibers exhibited a linearly increasing trend on the basis of an increase in ultimate bearing capacity of the range of reinforcement amounts utilized.

Yetimoglu T Salbas O; [2003] A study was undertaken to research the shear strength of sands reinforced with randomly distributed separate fibers by carrying out direct shear tests. The result of the fiber reinforcement content of the shear strength was investigated. The results of the tests indicated that peak shear strength and initial stiffness of the sand weren't affected significantly by the fiber reinforcement. The horizontal displacements at failure were also found comparable for reinforced and unreinforced sands below a similar vertical normal stress. Fiber reinforcements, however, may reduce soil brittleness providing smaller loss of post-peak strength. Thus, there appeared to be an increase in residual shear strength angle of the sand by adding fiber reinforcements.

Chandra et al.; [2008] have reinforced the three types of soil clay, silty and sand with polypropylene fiber of 0.3mm diameter. The fibers were cut into pieces of 15, 25, and 30mm in length and aspect ratio of 50, 80 and 100 respectively and with percentage of0.75, 1.5, 2.25 and 3 by dry weight of soil. The static triaxial test of unreinforced and reinforced soil was conducted. Their result shows that the uniaxial compressive strength is 3.824, 4.836 and 9.712 MPa respectively.

Using Fiber Reinforcement:

Maher and gray [1990] Have reinforced the coarse sand of nine varieties at Cu=1 to 4, D50=0. 09to0.6mm, 100% wet content with rubber (dia=1.1mm, ar=20, fl=22mm), glass (dia=0.3mm, ar=60, 08,125, fl=45mm), reed fiber (dia=0.3, ar=20, f=18, 24,38mm) Their Drain triaxial tests shows that low modulus fibers (rubber) contribute very little to strength despite higher interface friction. Failures surface is plain and adjusted at (45+Φ/2). An increase in particle sphericalness is higher in crucial confining pressure and lower fiber contribution. Higher aspect ratio resulted lower confining pressure and increasing shear strength.

Michalowski and Zaho [1996] have reinforced the dry sand (with Cu=1.52 and D50=0. 89) with polyamide monofilament and steel fibers (dia0.3,0.4 mm aspect ratio 85 and 180, fiber length and content 25 and 0.5% respectively). The triaxial result shows that the addition of steel fibers increases the peak stress by 20% and presence of fibers in habited the sample dilation and made sample stiff, before reaching the failure.

Bauer and Fatani; [1991] Have studied on silt sand (with Cu=5, D50=0.9,c=10kN/m2, Φ=470 at optimum moisture content) reinforced with steel fiber (rigid, dia=3mm,fl=40mm,,random) an copper(flexible , dia= 0.8mm ,fl=70mm,5,6 and 32 fibers aligned) They investigated the direct shear test and pull out test at modified proctor density test of 2.08t/m3 and moisture content of 8.9%, Φ=370 and δ=23°. The result shows that the residual strength of composite is 200th to 300th above unreinforced soil and well graded soil provide highest anchorage capacity

Fatani et al. [1991] Have studied on the silt sand with Cu=5 D50=0.9, c=10kN/m2, Φ=470 and reinforced with monofilament fiber of 70mm long, orientated (to the shear plane at 450 to 900) and random, number varies from 5to 32. The Drained direct test was done at modified proctor dry density γ = 20.8kN/m3 and optimum wetness content 8.9%, orientation of fiber is perpendicular to shear plane. The test result shows that fiber placed parallel test plane of direct shear box caused reduction in shear strength. In at random place, only 10-20% fibers cross the shear plane is really impart the strength.

Charan [1996] Has studied on silt sand to coarse sand (D50= 0.06-0.5mm) reinforced with polypropylene (dia=0.3 mm, ar=50to 125, fl=15 to 37, fc= 0.5 to 3%) and natural fibers coir and bhabar (ar = 50 to 100 fl= 15 to 37 mm, fc=0.5 to 3%). In this triaxial and CBR test were done to check the failure of composite. Triaxial result shows that confining pressure less than critical confining i.e1.2, strength of composite is un-affected by improving the density of composite. The CBR value is improved by 2 times at fiber content at 1.5%.

Wasti and Butun [1996] Have reinforced the sand soil (with Cu=3.995, Cc=1.132, D60 = 0.819 mm c= 6.98 ,Φ=47.8°) with polypropylene (30×50 mm small, 50×100mm big size and opening 10×10 mm 50mm long fiber by cutting mesh. They were conducted Laboratory model test on a strip footing 50mm (width) x 250 mm (length) supported by sand and randomly distributed polypropylene fiber and mesh element. Results indicate that reinforcement of sand caused an increase in the ultimate bearing capacity values and settlement at ultimate load. The big mesh size is superior to other and increases in ultimate bearing capacity.

Lindh and Eriksson [1990] Have reinforced the sand (Cu= 3.5 and D50=0.5mm) with monofilament polypropene fiber at fiber content of 0.25% and 0.5%. They were conducted a field experiment by inserting a reinforced sand layer on the present road surface for field experiment. Their result shows that no rutting is taken place.

2.2.3 Bearing capacity improvement of pond ash /fly ash using reinforcement:

Pond ash is an associate industrial waste having low bearing capacity and high settlement. Previously a number of scientists researched regarding the project. They tried to enhance the characteristics of pond ash by lime stabilization method or by reinforcing it by various Geotextiles.

M.V.S.Sreedhar et al., [2011] This project considers the application of geosynthetic reinforced pond Ash as an overlay on soft soils to act as sub-grade of a pavement. At intervals reinforced type, the reinforcement is provided in fabric type yet as in fiber type. The fiber is derived from a similar geotextile that is used in fabric type specified, the role of fabric property of the reinforcement is eliminated and the focus is only on its form. The results indicated that, the CBR characteristics of reinforced pond ash are higher than un-reinforced pond ash. Among the reinforced pond ash, the reinforcement in fabric type is more effective than that in fiber form. The result of soaking on reinforced pond ash is additionally studied.

Goutham Kumar Pothal et.al; [2007], Have conducted triaxial and load tests on reinforced pond ash and according improvement in bearing capacity. Bera et.al., Have reported conducting laboratory load tests on pond ash at its MDD, reinforced with one variety of jute geotextile and 3 types of plastic Geotextiles. They reported that, ideal depth of placement was 0.255B, wherever

B is the dimension of the model footing that resulted during a most improvement of 34th within the bearing capacity of reinforced pond ash. The modification in strength due to completely different compaction, controlling parameters, like layer thickness, compaction energy, tank size, wetness content, mould area, and relative density on the dry unit weight of pond ash are obtained.

Kumar et.al; [1999] Gives the results of laboratory investigations conducted on loose sand and pond ash specimens reinforced with arbitrarily distributed polyester fibers. The test results reveal that the inclusion of fibers in soils will increase the height compressive strength, CBR value, peak friction angle, and plasticity of the specimens. It's complete that the optimum fiber content for both loose sand and pond ash is approximately zero. 3 to 0.4% of the volume unit weight.

Sharan A.,S. P. Singh; [2011] Conducted a series of CBR tests on reinforced pond ash. They reported an increasing bearing resistance with the fiber content. However, the rate of increase of strength with fiber content is not uniform. At low strain levels the bearing resistance is found to remain almost constant with fiber content. However, at higher strain level the bearing resistance is found to increase substantially to increase in fiber content. This shows that to mobilize the strength of the fiber higher strain is required further more; it is observed that for a given compacted density an increase in fiber content results in decrease of initial stiffness whereas the failure strain increases. This indicates that inclusion of fiber gives ductility to the specimens. It can further be noticed that reduction in the post peak strain of a reinforced sample is comparatively lower than the unreinforced sample.

2.2.4 SCOPE OF THE PRESENT WORK:

The literature review shows that a limited work has been done to investigate the effectiveness of reinforcements in improving the load carrying capacity of the compacted pond ash. Keeping this in mind a series of laboratory tests were conducted to investigate:

1. The effect of reinforcement size and placement position on the CBR value of the compacted pond ash.
2. The effect of water content on the CBR value of compacted sand and the effect of the over lain sand layer in improving the CBR value of compacted pond ash.

CHAPTER -3

EXPERIMENTAL WORK AND METHODOLOGY

3. INTRODUCTION:

Experiments were done to determine various properties like specific gravity,grain sizeanalysis and geotechnical characteristics like maximum dry density, optimum moisture content, cohesion value, internal angle of friction of pond ash. For sand max and min dry density, bearing ratio for different relative density. Then the CBR of pond ash was determined by performing tests for both unsaturated conditions. Then the same procedure was repeated on saturated ash sample overlain by a sand layer and change in behavior of the saturated pond ash is observed. The following experiments were performed serially to observe the change in behavior of pond ash in different conditions.

3.1 MATERIAL USED:

POND ASH:
The sample passing through the sieve of 2mm diameter was used in experiments. Pond ash was dried in the oven at 105˚C-110˚C and kept in an airtight container for further use.

SAND:
The specific gravity of soil is the ratio between the weight of the soil solids and weight of an equal volume of water. It is measured with the help of a volumetric flask in a very simple experimental setup where the volume of the soil is found out and its weight is divided by the weight of an equal volume of water. The test results are obtained 2.605 specific gravity of sand.

3.1.1 DETERMINATION OF TENSILE STRENGTH OF REINFORCING MATERIAL:

Tensile strength test was carried out on reinforcing materials. The properties obtained are given below in Table 3.2

Table 3.2 Tensile strength of reinforcing material

SAMPLE ID	GI SHEET	PVC MESH
Displacement at peak (mm)	5.368	82.42
Strain at peak (%)	15.34	235.5
Load at peak (kN)	0.6029	0.021
Stress at peak (Mpa)	25.37	0.8030
Displacement at break (mm)	7.602	127.4
Strain at break (%)	21.72	364
Load at 0.2% yield (kN)	0.288	0.0085

Stress at 0.2% yield (Mpa)	12.15	0.325
Young's modulus	1082	39.16

3.1.2 SPECIFIC GRAVITY TEST:

The experiment is performed according to the above IS code procedure. The specific gravity of pond ash was determined by density bottle and illustrated. Specific gravity of pond ash was found as per IS: 2720 (Part III) 1980 and obtained as 1.95.

3.1.3 DETERMINATION OF GRAIN SIZE DISTRIBUTION:

For determination of grain size distribution, the pond ash was responded to test sieve having an opening size 75µ. Sieve analysis was conducted for coarse particles as per IS: 2720 part (IV), 1985 and hydrometer analysis was conducted for finer particles as per IS: 2720 part (IV). The share of pond ash passing through 75µ sieve was found to be 36.28%. Therefore the particle size of pond ash ranges from fine sand to silt size. Uniformity coefficient (Cu) and coefficient of curvature (Cc) for pond ash was found to be 8.33 and 0.75 respectively and indicating uniform gradation of samples. The grain size distribution curve of pond ash is part in Fig. 3.2. Coefficient of uniformity and co-efficient of curvature were detected using the subsequent formula.

Coefficient of uniformity, Cu = D60 / D10

Coefficient of curvature, Cc = (D30) 2 / (D60 * D10)

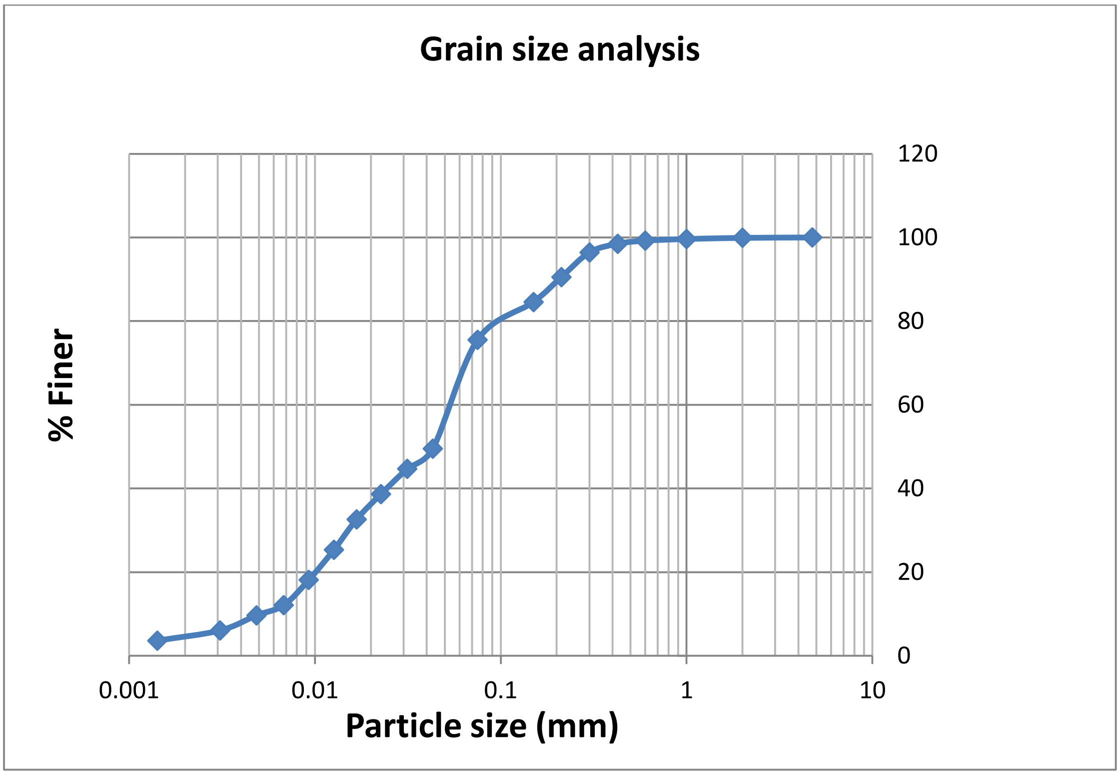


Fig 3.2 Grain size analysis of pond ash

3.2 DETERMINATION OF ENGINEERING PROPERTIES:

3.2.1 Determination of OMC & MDD of pond ash:

The moisture content, dry density relationships were found by using compaction tests as per IS: 2720 (Part 7) 1980. For this test, pond ash was mixed with needed quantity of water and the wet sample was compacted in proctor mould either in 3 or 5 equal layers using standard proctor rammer of 2.6 kg and modified proctor rammer of 4.5 kg severally. The moisture content of the compacted mixture determined as per IS: 2720 (Part II) 1973. From the dry density and moisture content relationship (graph), optimum moisture content (OMC) and maximum dry density (MDD) were determined. Similar compaction tests were conducted with varied compactive energy and therefore the corresponding OMC and MDD were determined. This was done to review the result of compactive energy on OMC and MDD. The test results are presented in Table 3.3 and graphs were plotted.

Light compaction maximum dry density =1.173 g /cc, OMC=25.5

Heavy compaction maximum dry density = 1.276 g/cc, OMC=22.1

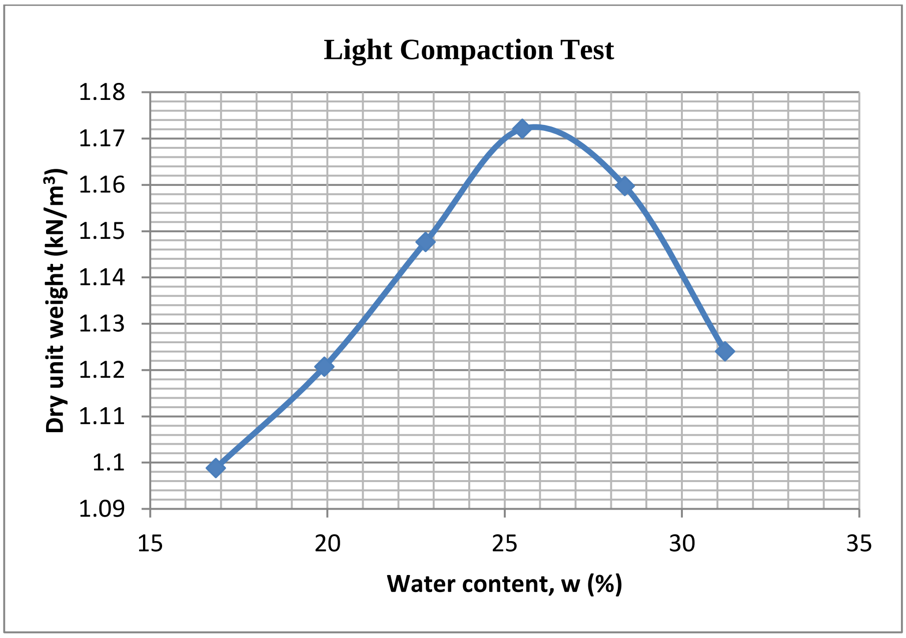


Fig-3.3 Water content- Dry unit weight of pond ash for Light compaction test

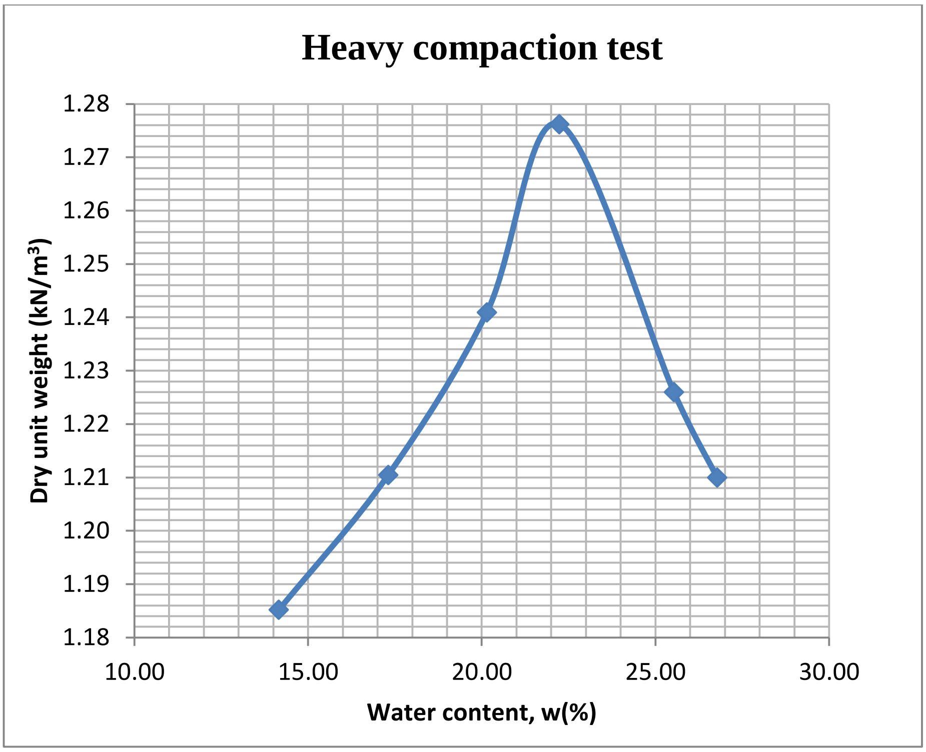


Fig-3.4 Water content- Dry unit weight of pond ash for Heavy compaction test

3.2.2 DETERMINATION OF MINIMUM DRY DENSITY OF SAND:

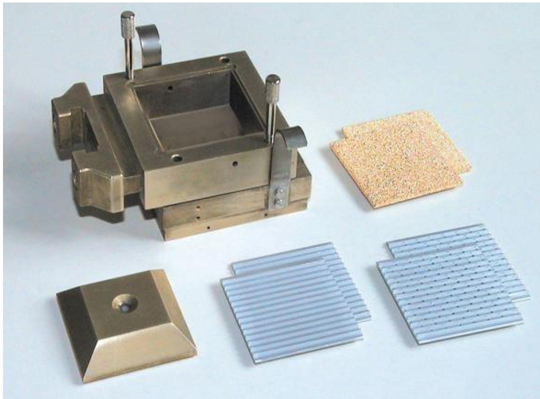
Sand containing particles smaller than 9.50 mm should be placed as loosely as possible within the mould by pouring the sand through the spout during a steady stream. The spout should be adjusted so the height of free fall of the sand is usually 25 mm. While pouring the sand the pouring device should be moved in a spiral motion from the outside towards the center to make a sand layer of uniform thickness without segregation. The mould should be filled approximately 25 mm above the top and levelled with top by. Making one continuous pass with the steel straight edge. If all excess matter is not removed, an additional continuous pass should be made. Great care shall be exercised to avoid jarring the mould during the entire pouring and trimming operation. The mould and the sand should be weighed and the mass recorded. As per experimental investigations as obtained minimum dry density of sand is γ_d **(max) 14.1 kN/m^3**

3.2.3 DETERMINATION OF MAXIMUM DRY DENSITY OF SAND:

The guide sleeve should be assembled on top of the mould and the clamp assemblies tightened so that the inner surfaces of the walls of the mould and therefore the sleeve are in line. The lock nuts on the two set screws equipped with them should be tightened. The third clamp should be loosened, the guide sleeve removed, the empty mould weighed and its mass recorded. The mould should then be filled with the thoroughly mixed. The mould filled for the determination of minimum density may also be used for this test.The guide sleeves should be connected to the mould and the surcharge base plate should be placed on the sand surface. The surcharge weight should then be lowered on the base-plate using the hoist in the case of the 15 cm mould.The mould should be fixed to the vibrator deck. The vibrator control should be set at and the amplitude and also the loaded sand specimen should be vibrated for 8 minutes. As per experimental investigations as obtained minimum dry density of sand is **γ_d (max) 16.1 kN/m^3**

3.2.4 DETERMINATION OF SHEAR PARAMATERS:

This test is performed to determine the consolidated-drained shear strength of a sand of loose soil. The shear strength is one among the most necessary engineering properties of a soil because it is needed whenever a structure depends on the soil's shearing resistance. The shear strength is required for engineering situations like determining the stability of slopes or cuts, finding the bearing capacity for foundations and calculative the pressure exerted by a soil on a retaining wall. The c and Φ values of pond ash were known by direct shear test. It consisted of a box with a dimension of 60mm x 60mm x 50 mm depth. Specimen of size 60mm x 60mm x 25 mm was prepared at MDD and OMC and also at saturated condition and sheared with a constant strain for various normal stress. A graph is plotted between shear stress vs normal stress from that c and Φ values seen discovered. Direct shear test was conducted for the soil samples at light compaction density and heavy compaction density. Cohesion and angle of internal friction was found to be 0.11kg/cm^2 and 34˚ respectively. The shear strength parameters of the compacted specimens were determined from normal stress versus shear stress plots and it is given in Fig-3.5.



DIRECT SHEAR TEST MACHINE AND DIFFERENT PARTS OF SHEAR BOX

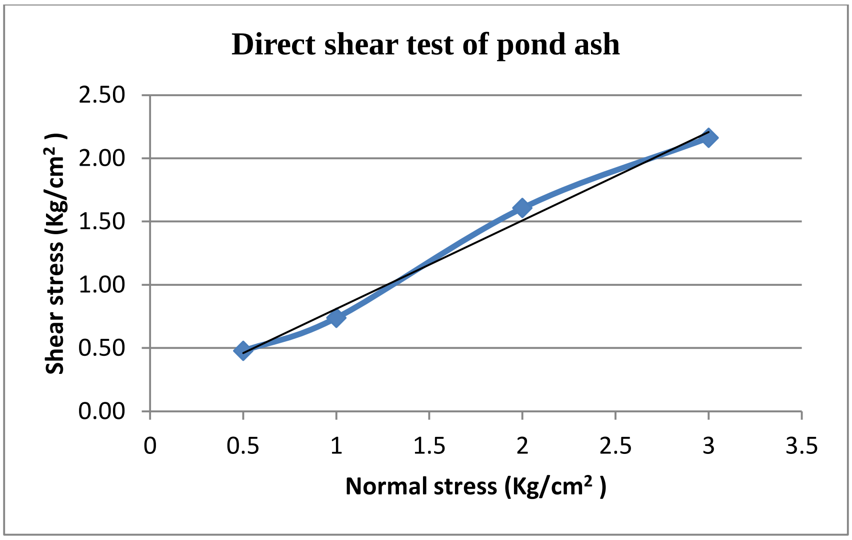


Fig-3.5 Normal stress- Shear stress of Pond ash

3.2.5 DETERMINATION OF CALIFORNIA BEARING RATIO:

3.2.5.1 CBR value of compacted pond ash:

The design of pavement includes the necessity of study of the properties of sub base and base of the soil. This includes the determination of strength and bearing capacity of the soil, this can be accomplished in the field by finding the CBR index of the soil. Whereas, the study necessitates the lab tests to be followed by field application, the following procedure determines the lab tests:

A cylindrical mould of dimensions 150 mm diameter, 175 mm height is used. At MDD and OMC the sample is prepared, over the sample spacer disc is placed and compacted with the hydraulic jack till the level of the spacer disc reaches the top of the mould. The whole set up is placed on the CBR testing machine. Now a surcharge simulating the field conditions is placed at the middle of the mould, and the load is applied with a movable base set up at a constant strain rate of 1.2 mm/min. The piston applying load was 50 mm diameter and the applied load was recorded till the 13 mm penetration depth achieved. To assess the stability of pond ash the above test was conducted on the unsoaked condition, according to IS 2720 (Part XVI) -1987. The graph plotted was shown in Fig 4.1.

3.2.5.2 CBR value of reinforced pond ash:

The CBR test as per IS 2720 (Part XVI) -1987 was conducted with different size of reinforcement from 5 to 15 cm uniformly varying at 2.5 cm along the diameter and CBR values of pond ash at different depths is studied (2.5cm, 5cm,7.5cm, 10cm). The two different kinds of reinforcement considered in the present study are Galvanized iron (GI) and Poly Vinyl chloride (PVC), and the graphs are shown in Fig 4.2 to 4.19.

3.2.5.3 CBR value of pond ash over lain by sand:

The CBR tests were done on compacted pond ash overlain by a layer of sand with varying relative density and with different thickness and it was observed that at 90% relative density of sand higher CBR value was observed, and the graph plotted was shown in Fig 4.39.

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Chapter 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION:

A series of CBR tests were conducted on compacted pond ash and compacted pond ash reinforce ether with grid reinforcements of G I & PVC are a layer of overlain sand. The position of reinforcements and the size were varied. Further the thickness of over lain sand layer and the relative densities of sand were varied and CBR test conducted. The test results are presented in the following sub sections.

4.1.1 Load deformation of compacted pond ash:

The load deformation behavior of pond ash compacted to either standard proctor densities are modified proctor densities as shown in fig 4.1 as the compaction energy increases the stiffens as well as the failure load increases the CBR value corresponding to 2.5 mm penetration and 5 mm penetration are found for samples compacted at standard proctor density and these values are for samples compacted at modified proctor density**.**

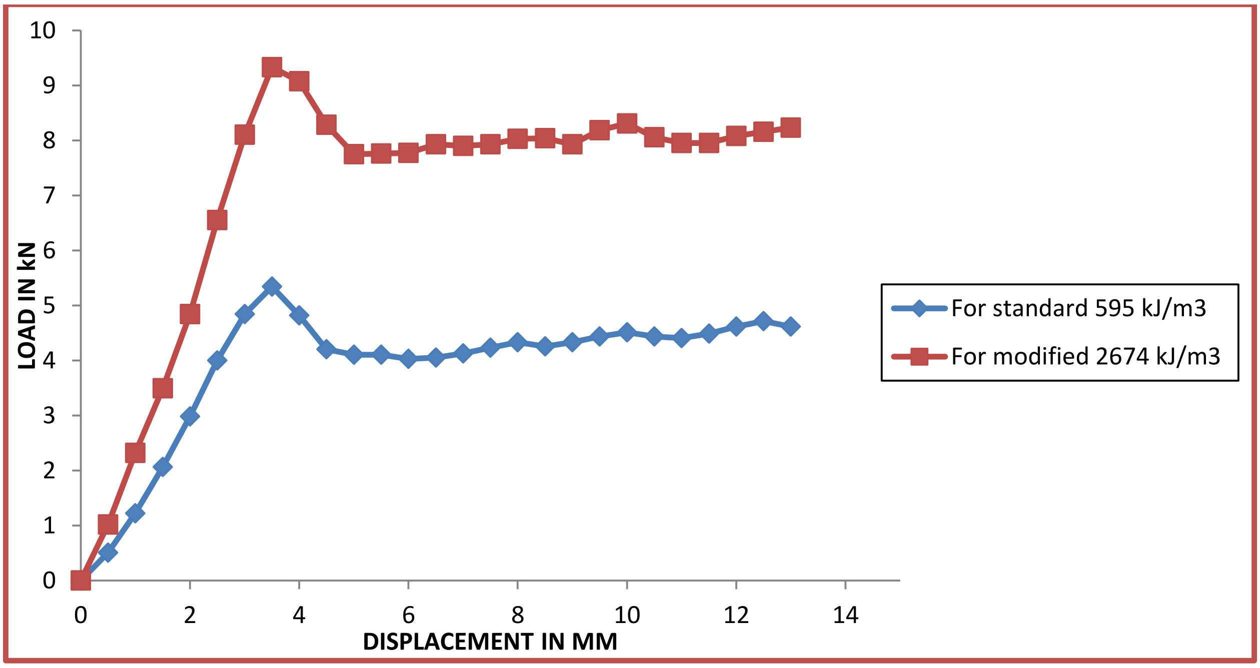

Fig 4.1 load deformation behavior of compacted pond ash

Table 4.1 CBR value of unsoaked compacted pond ash

Compaction energy	CBR value corresponding to 2.5mm penetration (%)	CBR value corresponding to 5mm penetration (%)
Light compaction	10.34	9.4
Heavy compaction	19.42	17.6

4.1.2 LOAD DEFORMATION BEHAVIOR OF COMPACTED POND ASH REINFORCED WITH G.I &P.V.C NETS:

A series of CBR tests were conducted in compacted pond ash specimens reinforced with GI& PVC nets. The diameter of the reinforcements were varied as 5 cm ,7.5cm,10cm, 12.5 cm& 15 cm and these reinforcement were placed either at 2.5 cm , 5 cm,7.5 cm&10 cm depth below the top surface. The pond ash sample were compacted either standard Proctor density or modified Proctor density. The reinforcements were net type made up of GI or PVC material the load deformation curves obtained for these test variables are given in Fig 4.2 to 4.16. Figures 4.3 to 4.9 shows the load deformation curves when different sizes of the reinforcements were placed at the depth of either 2.5 cm ,5cm,7.5cm &10cm respectively. It's seen at a given depth as the size of the reinforcement increase the stiffens of the load settlement was increased so as the failure load. Furthermore as the samples are compacted with higher load its failure load increase. However the strain at failure load is found to be almost same for the all sizes of reinforcement.it is seen that once the pond ash is reinforced G.I nets it carried higher load than PVC reinforced pond ash at comparable test conditions. This may be due to the higher stiffens of G.I net than PVC nets. Further the higher apertures sizes in G.I net makes possible an interaction between the pond ash particle below and above the net, which is absent in the PVC reinforcement. The aperture in PVC net is quite small prohibiting and interaction between pond ash below and above it thus acting as a separator.

For standard (compaction energy 595 kJ/m^3)

Reinforcement position: 2.5 cm from top

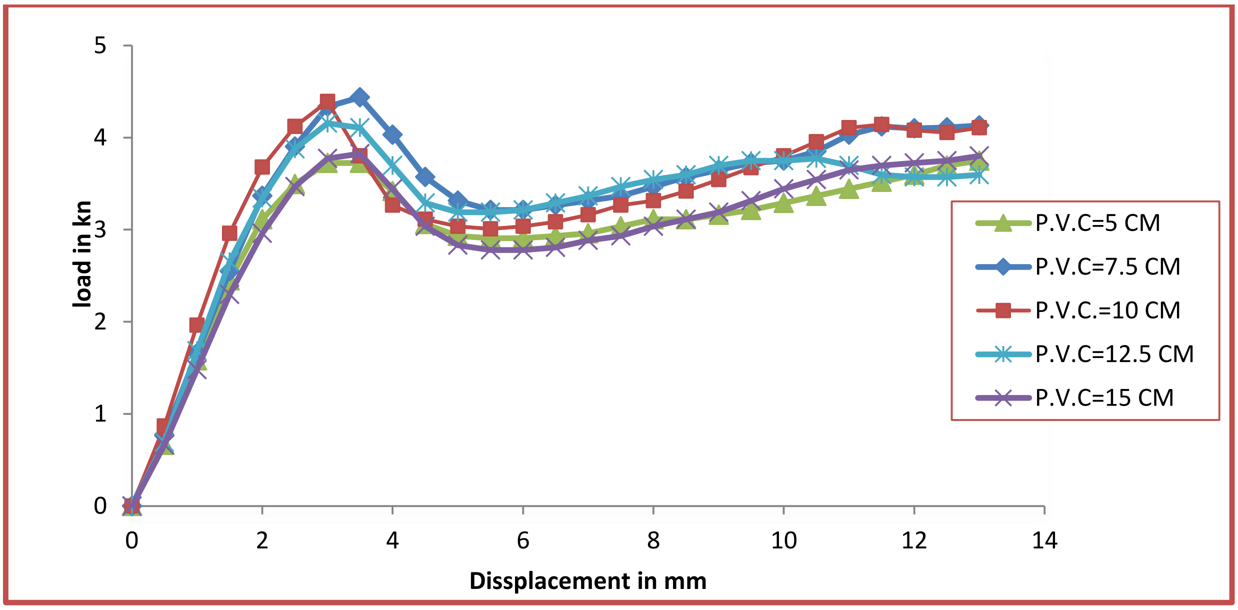

Fig 4.2 load deformation behavior of compacted pond ash (standard density) reinforced with PVC nets

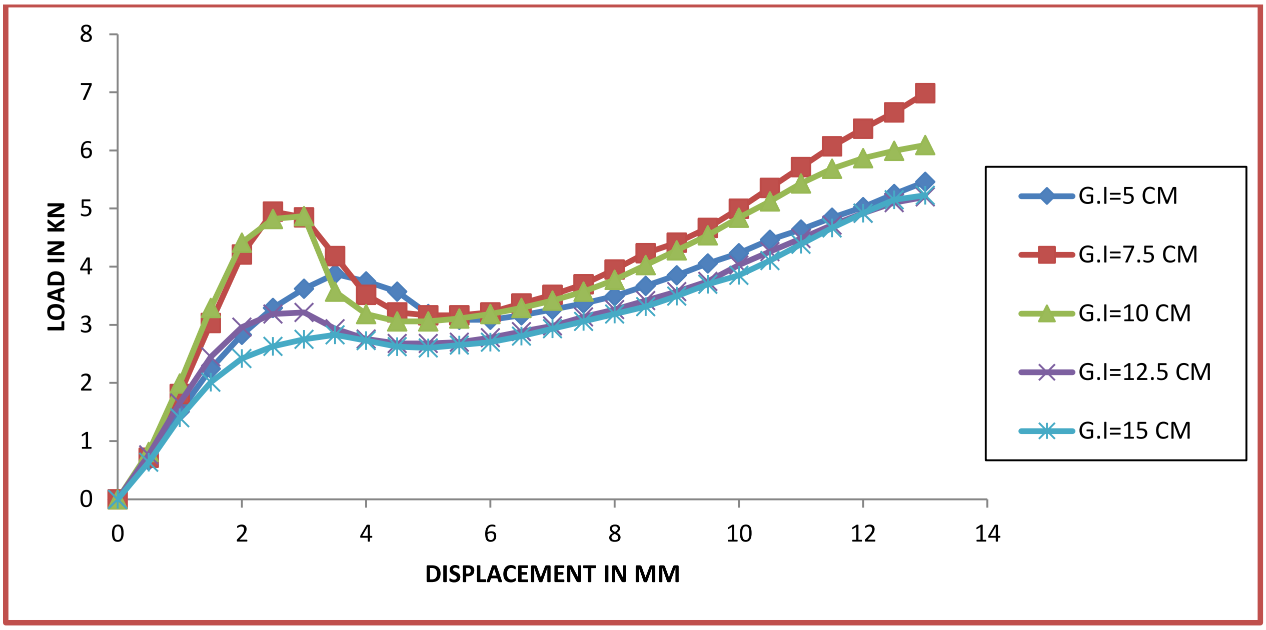

Fig 4.3 Load deformation behavior of compacted pond ash (standard density) reinforced with GI nets

Reinforcement position: 5 cm from top

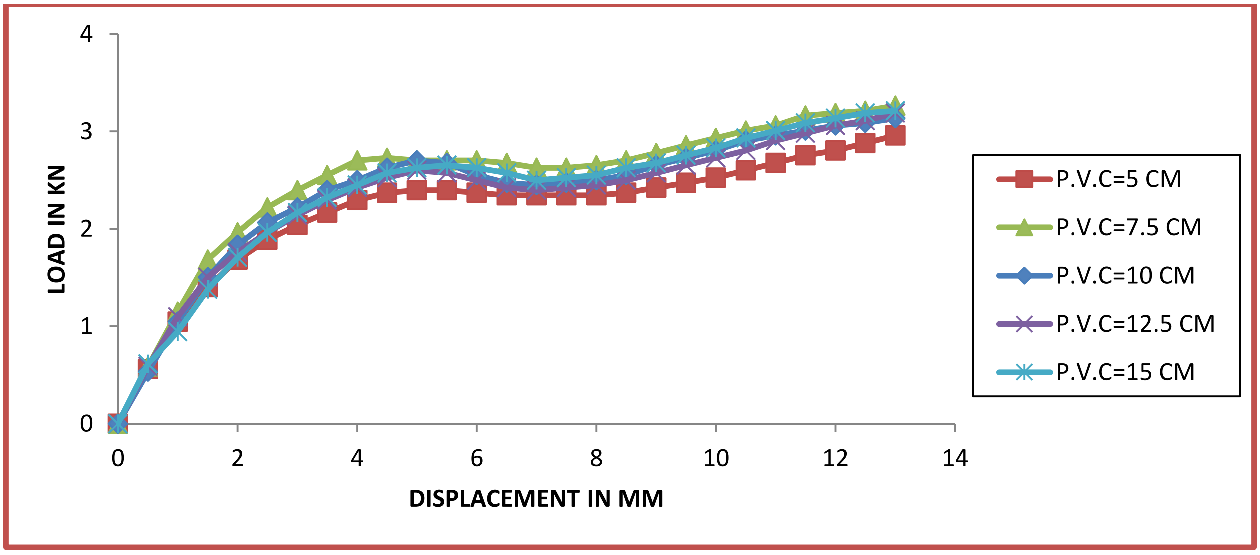

Fig 4.4 load deformation behavior of compacted pond ash (standard density) reinforced with PVC nets

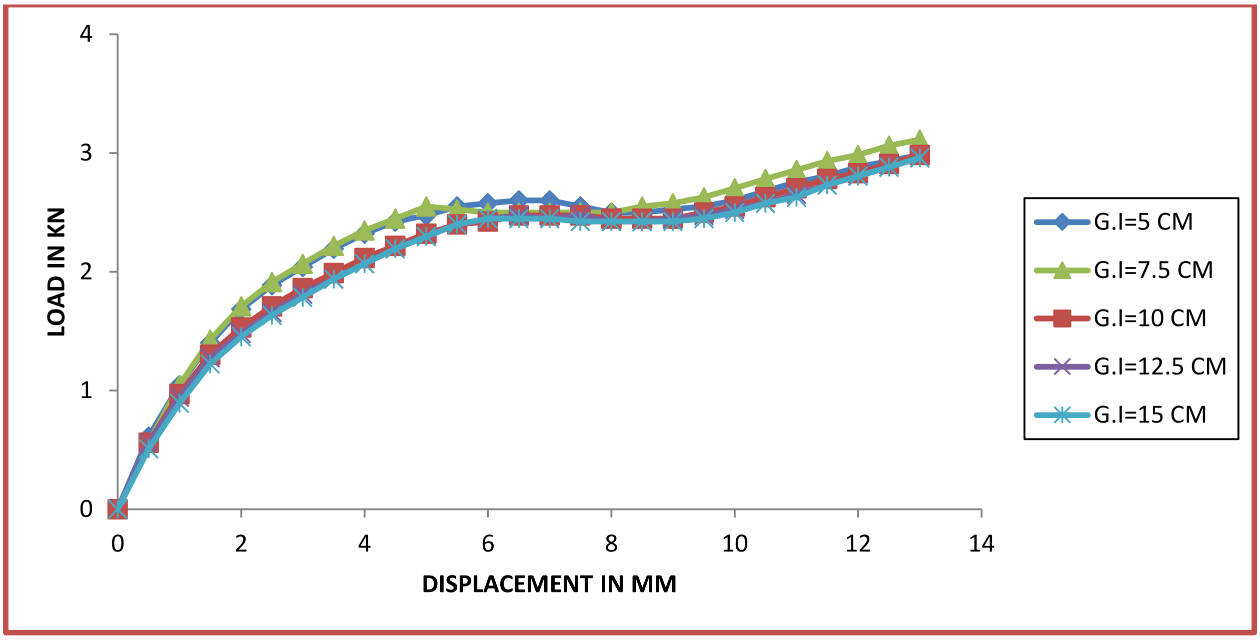

Fig 4.5 load deformation behavior of compacted pond ash (standard density) reinforced with G.I nets

Reinforcement position: 7.5 cm from top

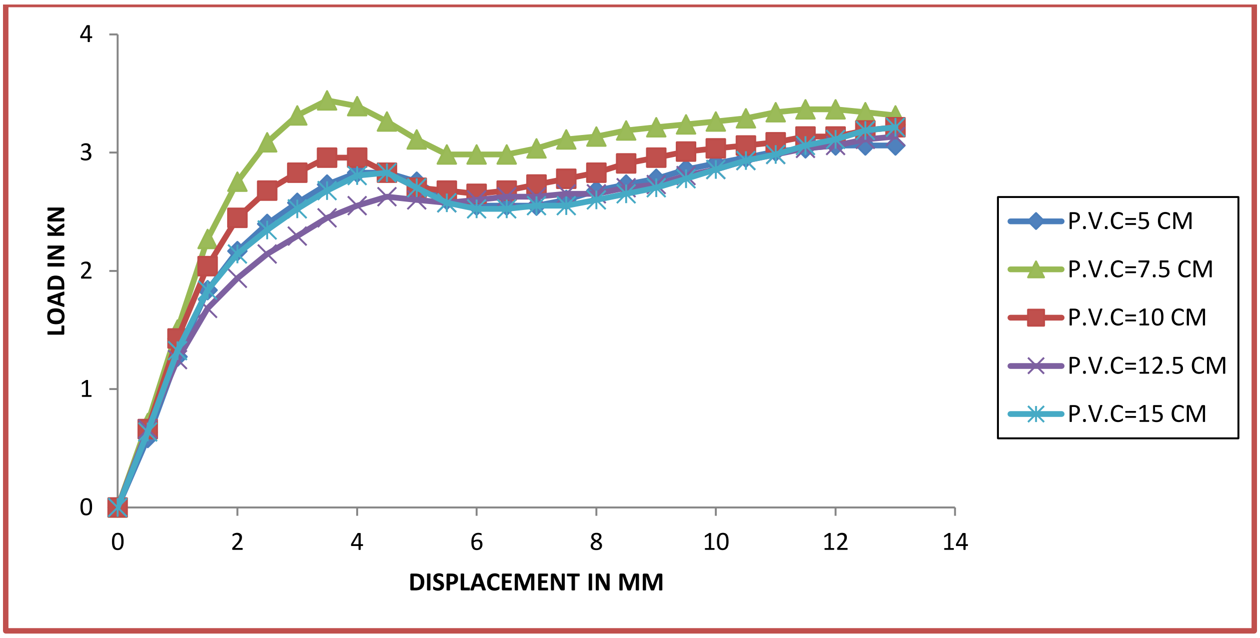

Fig 4.6 load deformation behavior of compacted pond ash (standard density) reinforced with PVC nets

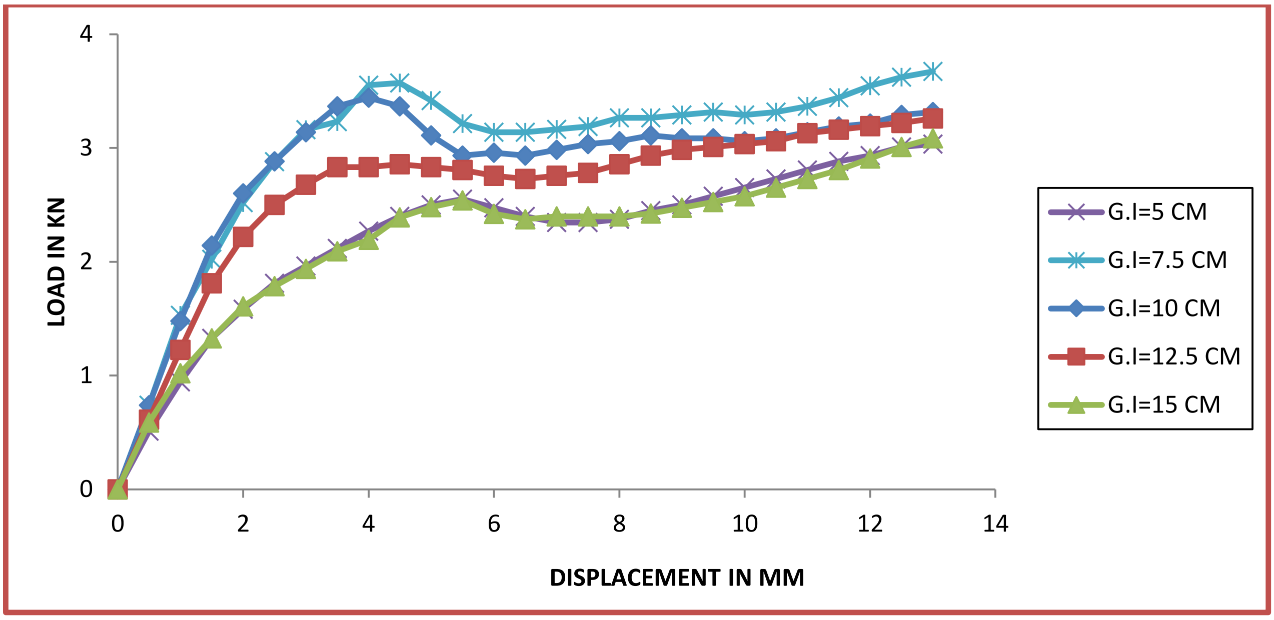

Fig 4.7 load deformation behavior of compacted pond ash (standard density) reinforced with G.I nets

Reinforcement position: 10 cm from top

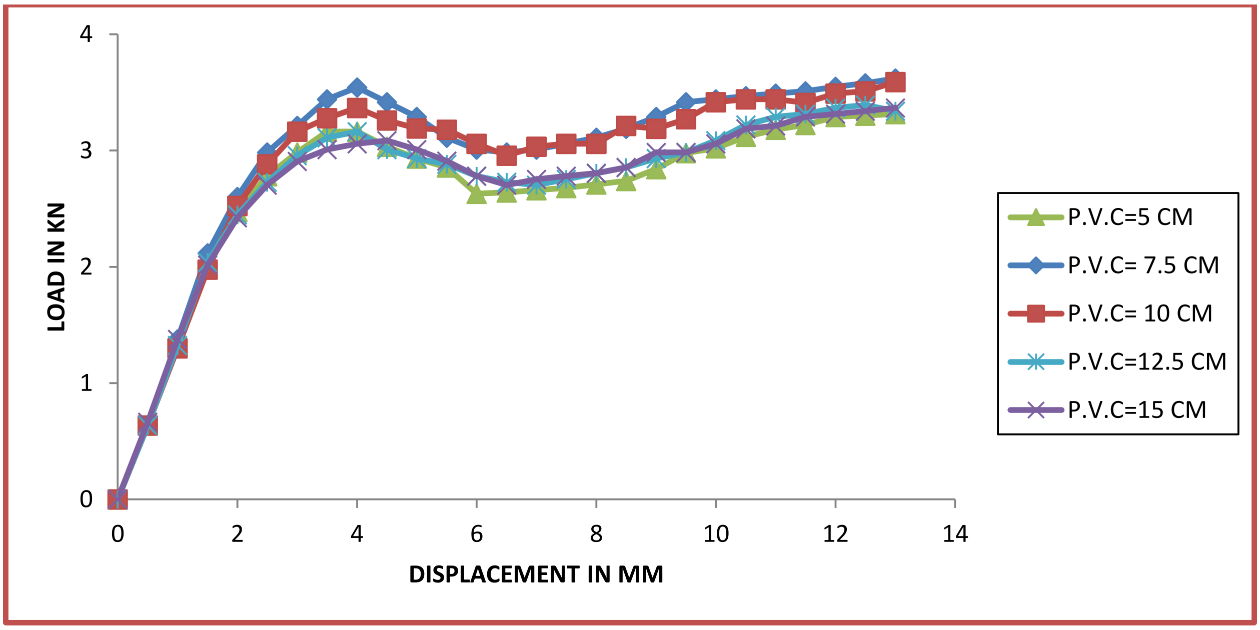

Fig 4.8 load deformation behavior of compacted pond ash (standard density) reinforced with PVC nets

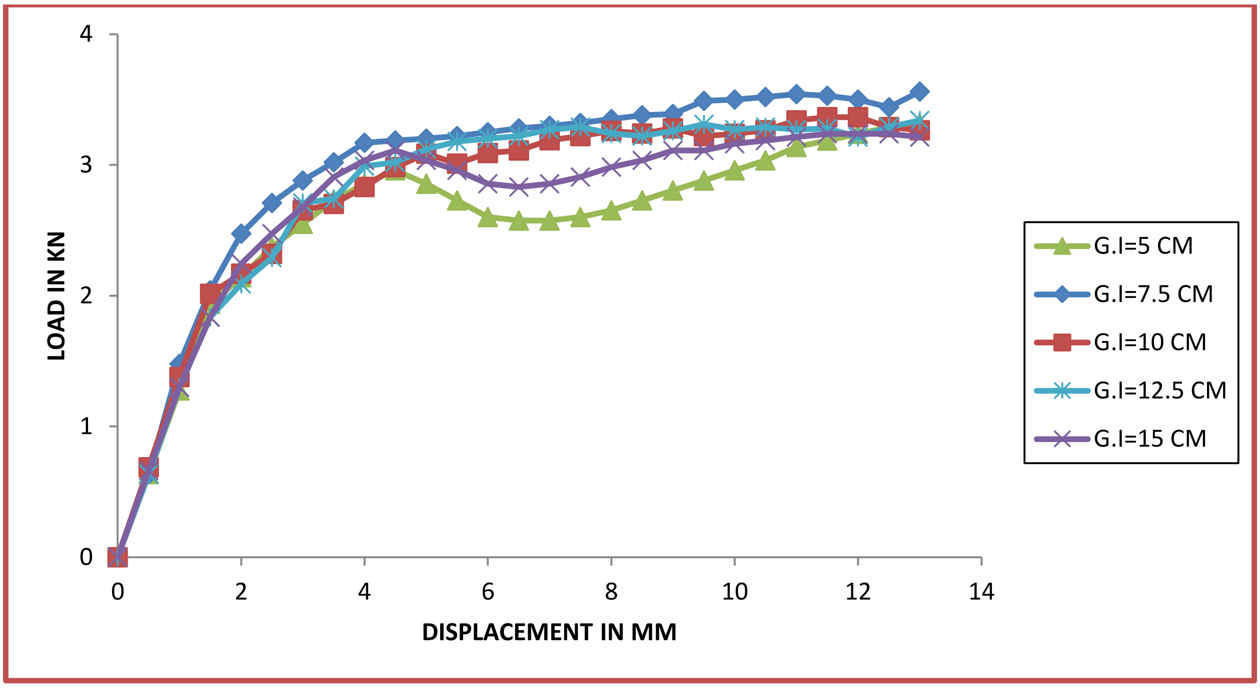

Fig 4.9 load deformation behavior of compacted pond ash (standard density) reinforced with G.I nets

FOR MODIFIED (COMPACTION ENERGY 2674 KJ/M^3)

Reinforcement position: 2.5 cm from top

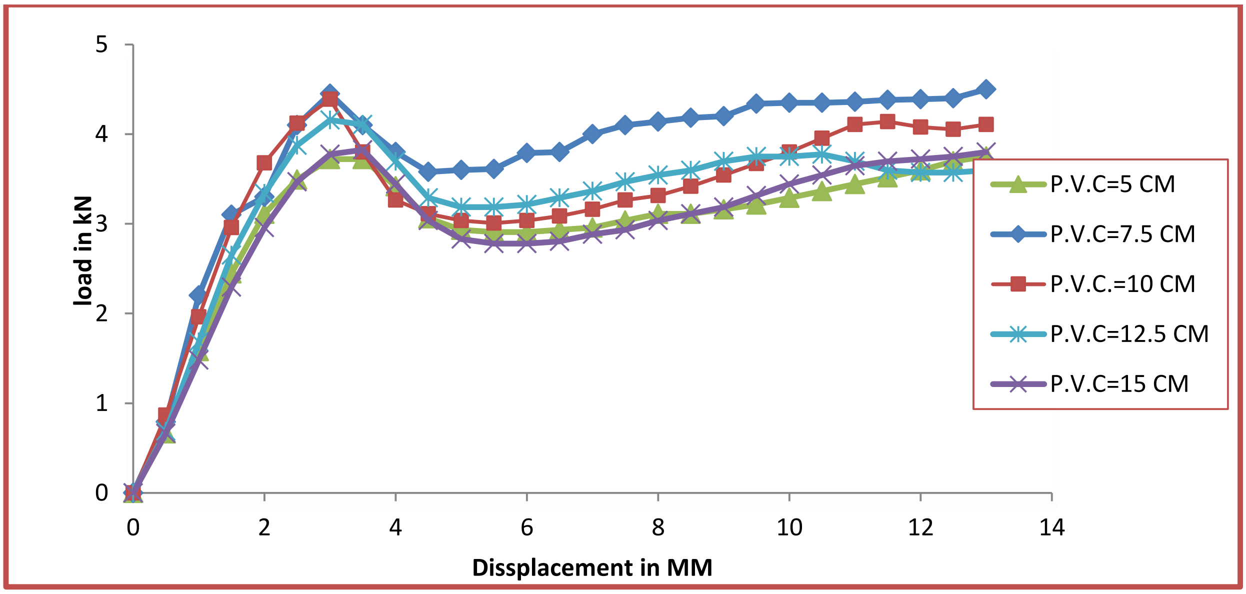

Fig 4.10 load deformation behavior of compacted pond ash (modified density) reinforced with PVC nets

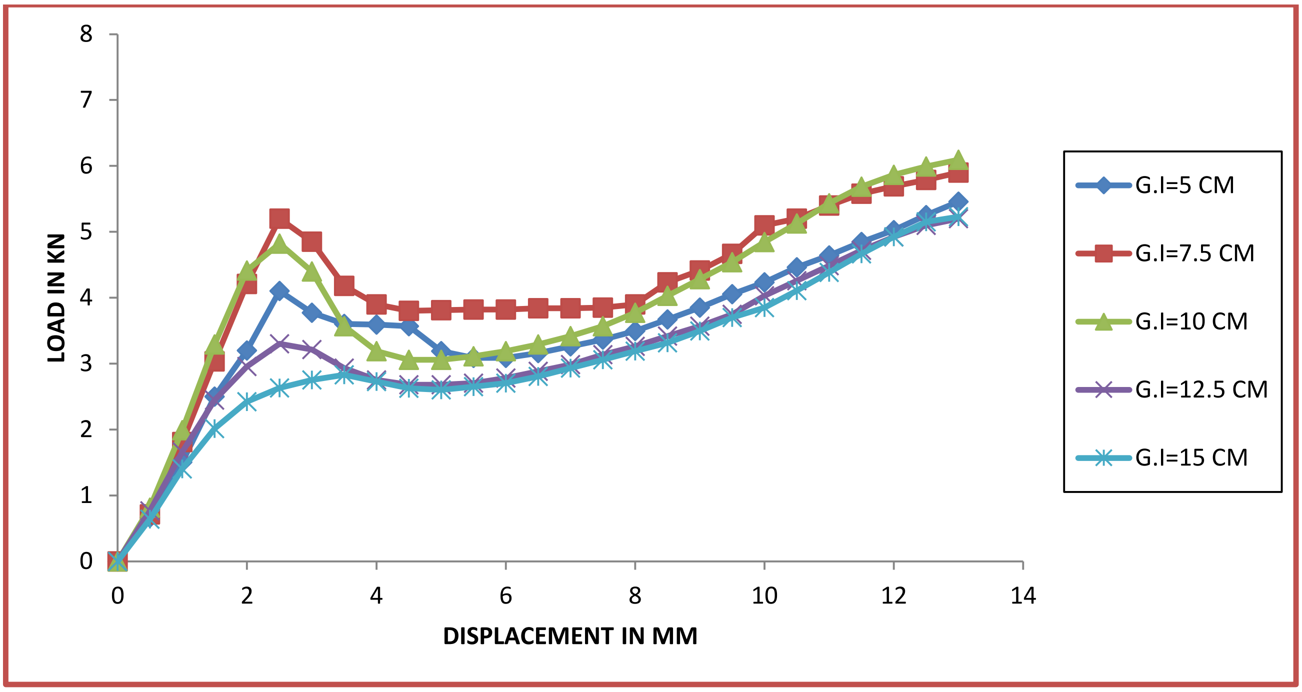

Fig 4.11 load deformation behavior of compacted pond ash (modified) reinforced with G I nets

Reinforcement position: 5 cm from top

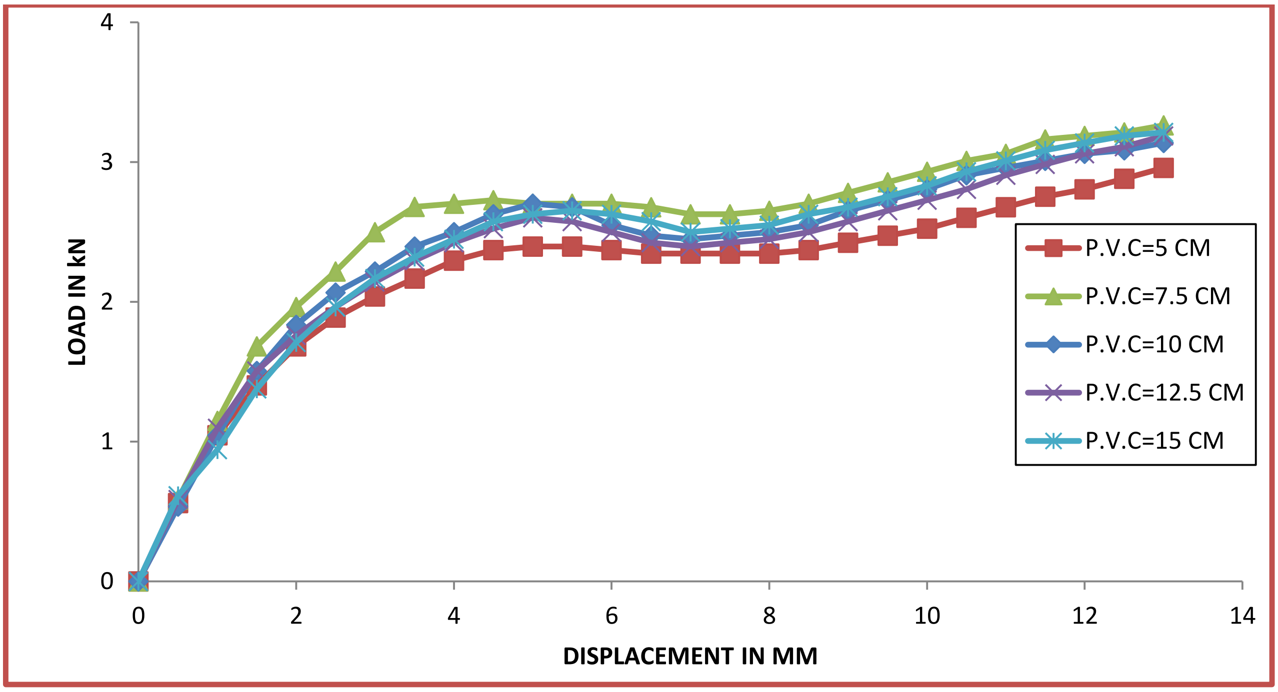

Fig 4.12 load deformation behavior of compacted pond ash (modified) reinforced with PVC nets

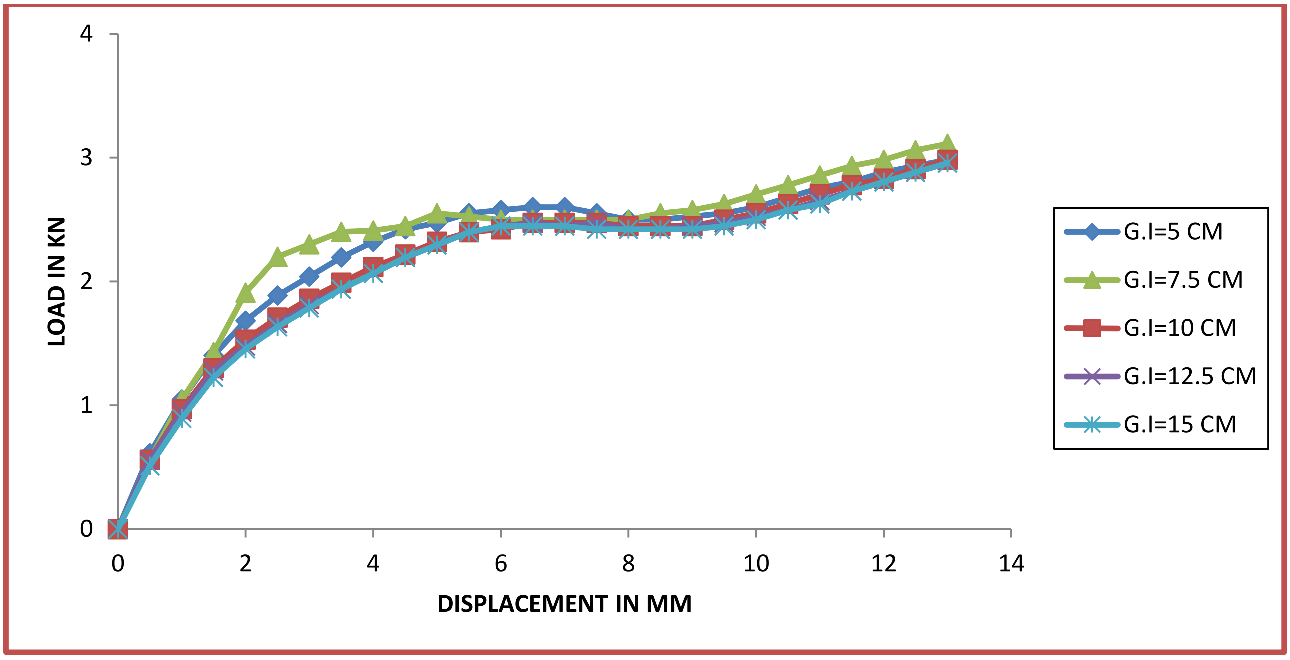

Fig 4.13 load deformation behavior of compacted pond ash (modified) reinforced with G I nets

Reinforcement position: 7.5 cm from top

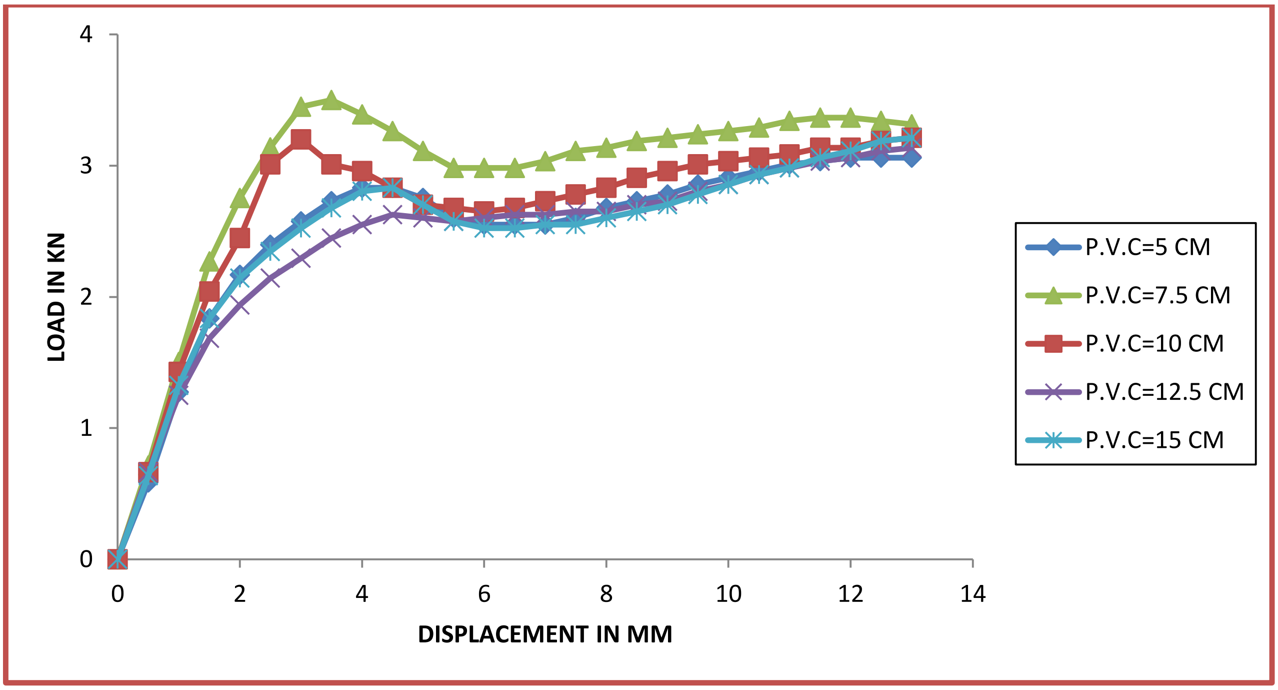

Fig 4.14 load deformation behavior of compacted pond ash (modified density) reinforced with PVC nets

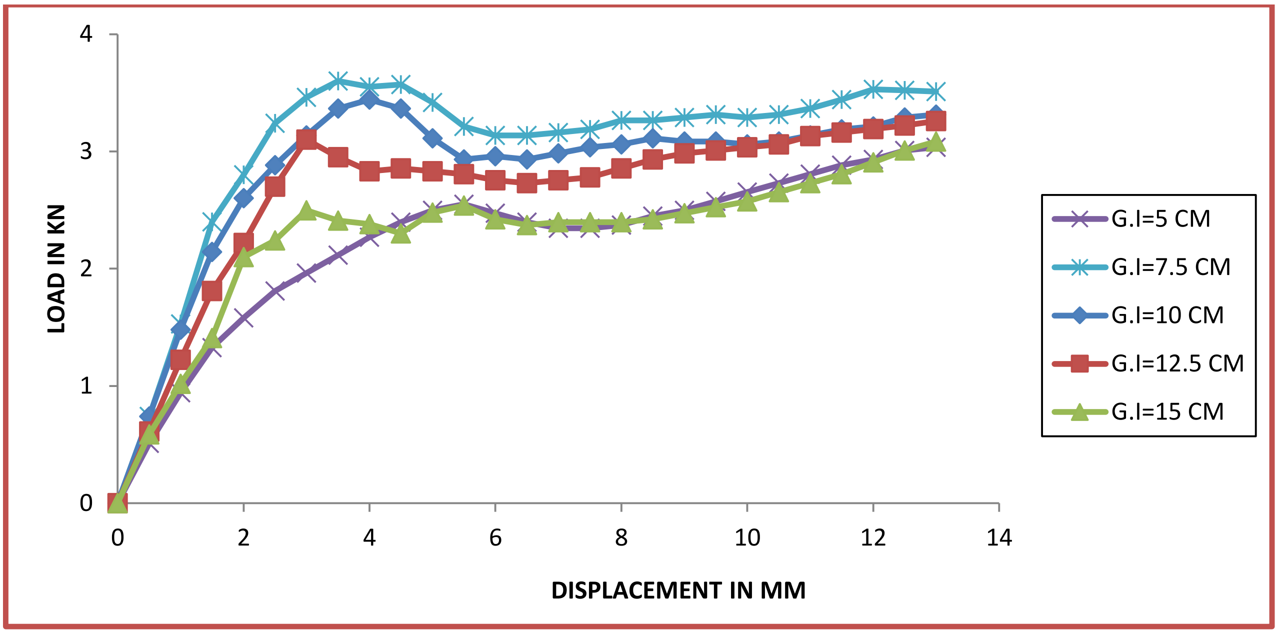

Fig 4.15 load deformation behavior of compacted pond ash (modified) reinforced with G I nets

Reinforcement position: 10 cm from top

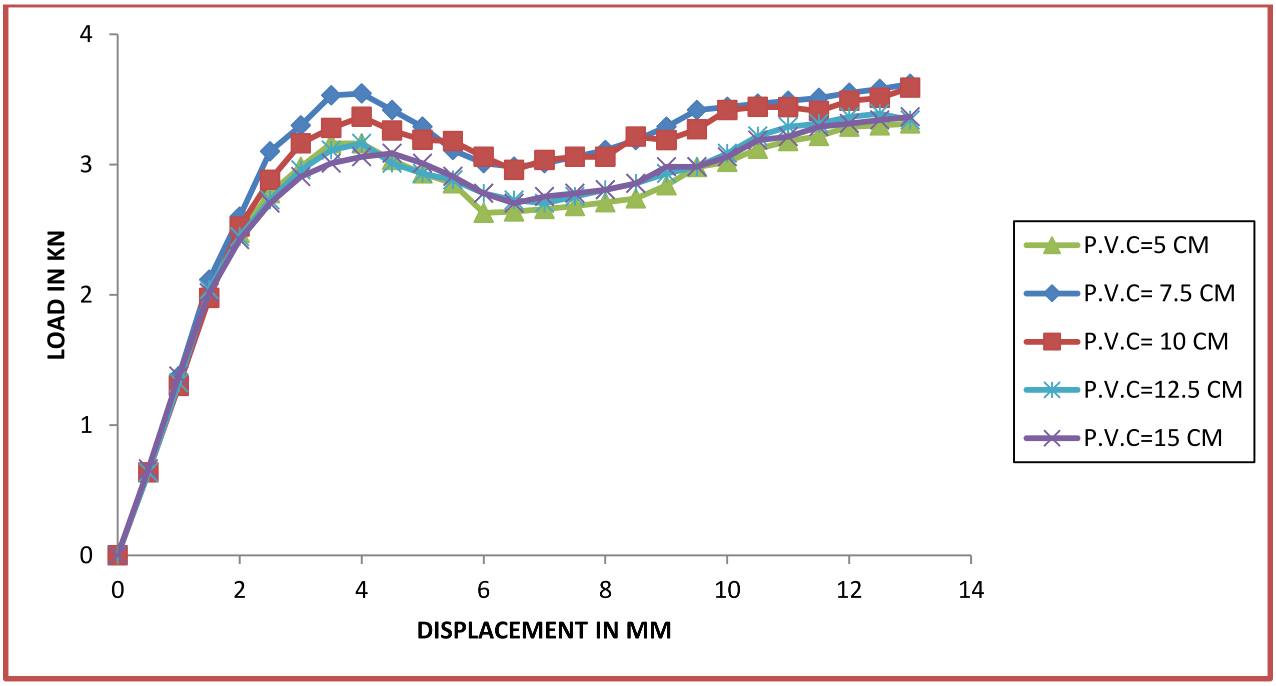

Fig 4.16 Load deformation behavior of compacted pond ash (modified density) reinforced with PVC nets

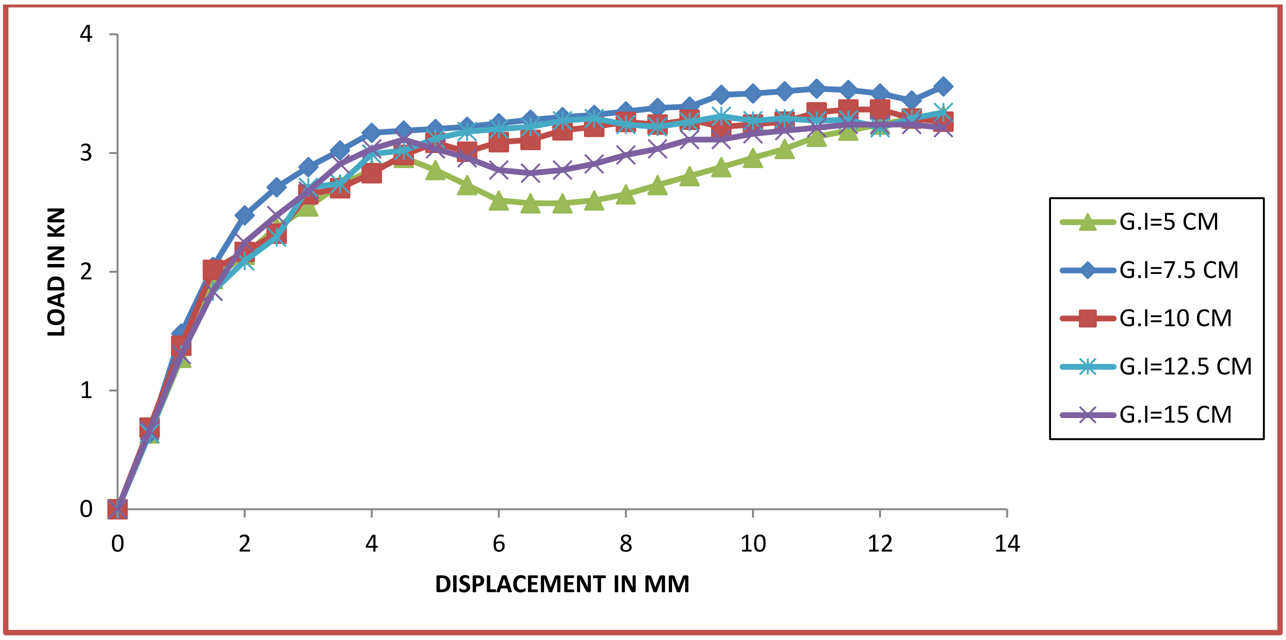

Fig 4.17 load deformation behavior of compacted pond ash (modified) reinforced with G I nets

4.1.3 EFFECT OF REINFORCEMENT SIZE ON CBR VALUE:

Variation of CBR value with depth of reinforcement for samples compacted at standard proctor density and reinforced with G.I reinforcements as shown in Figure. CBR values of pond ash increases with the inclusion of reinforcement. However when the depth of reinforcement greater than two times of diameter of footing shows no significant changes in CBR. It was observed that the CBR values of reinforced pond ash increases with increase in stiffness of reinforcement. The present work used two types of reinforcement namely PVC and GI. At the same reinforcement depth, inclusion of GI material found to give more CBR value than PVC irrespective of the test condition. Fig 4.18 to Fig 4.25 shows the variation of CBR value with position of reinforcement (compacted samples of both at standard density and modified density). Inclusion of reinforcement in compacted pond ash generally found to increases the CBR value and therefore strength subsentially.

For compacted at standard density

At 2.5 mm penetration:

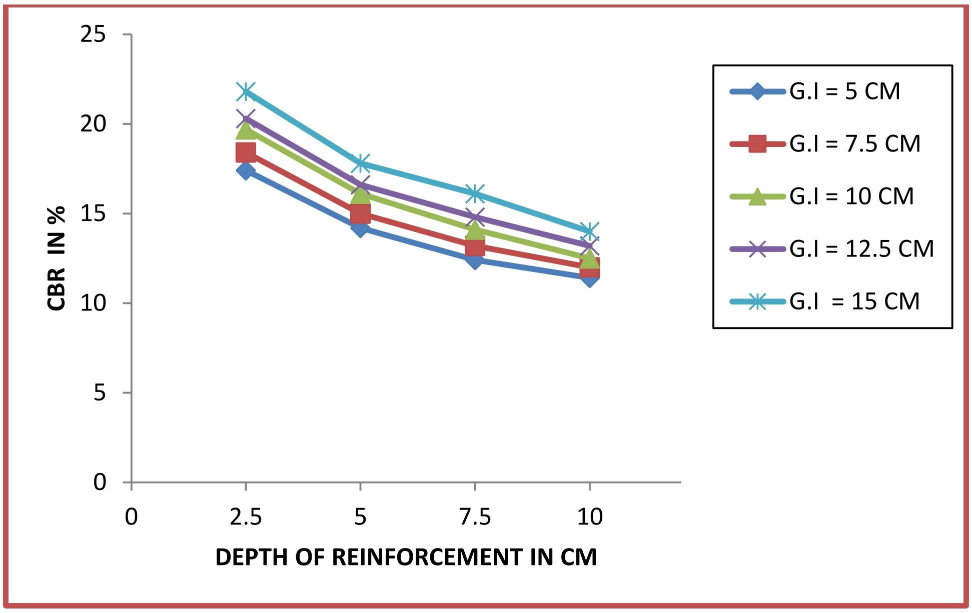

Fig 4.18 variation of CBR value with position of reinforcement (samples compacted at standard density)

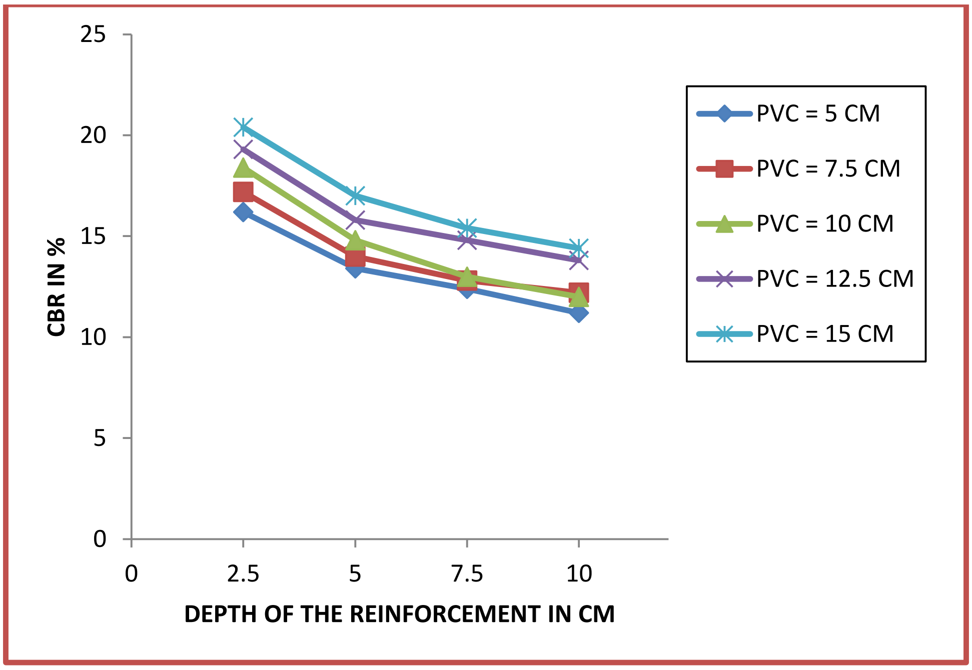

Fig 4.19 variation of CBR value with position of reinforcement (samples compacted at standard density)

At 5 mm penetration:

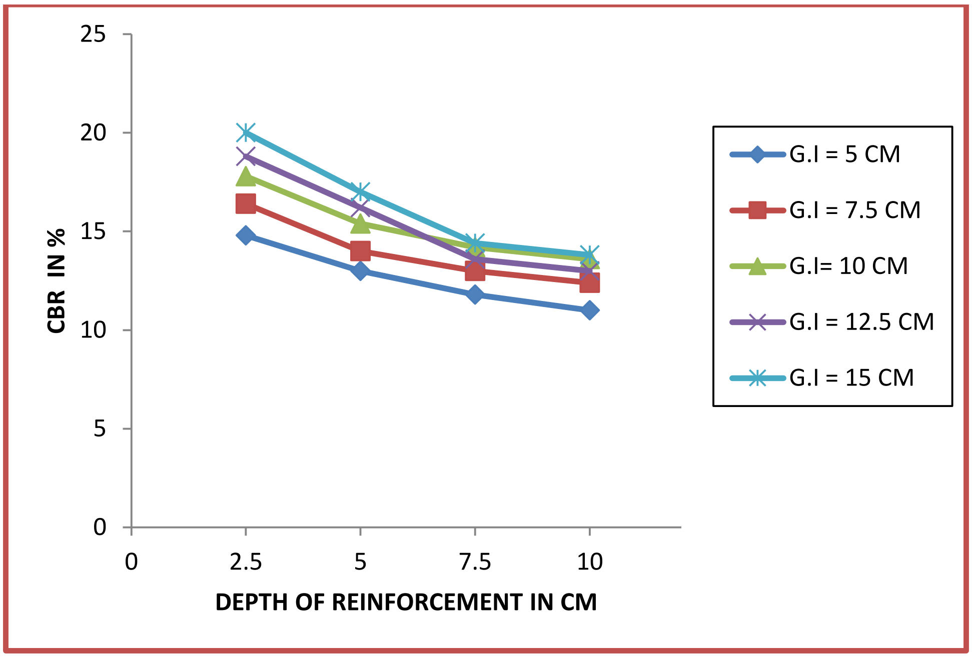

Fig 4.20 variation of CBR value with position of reinforcement (samples compacted at standard density)

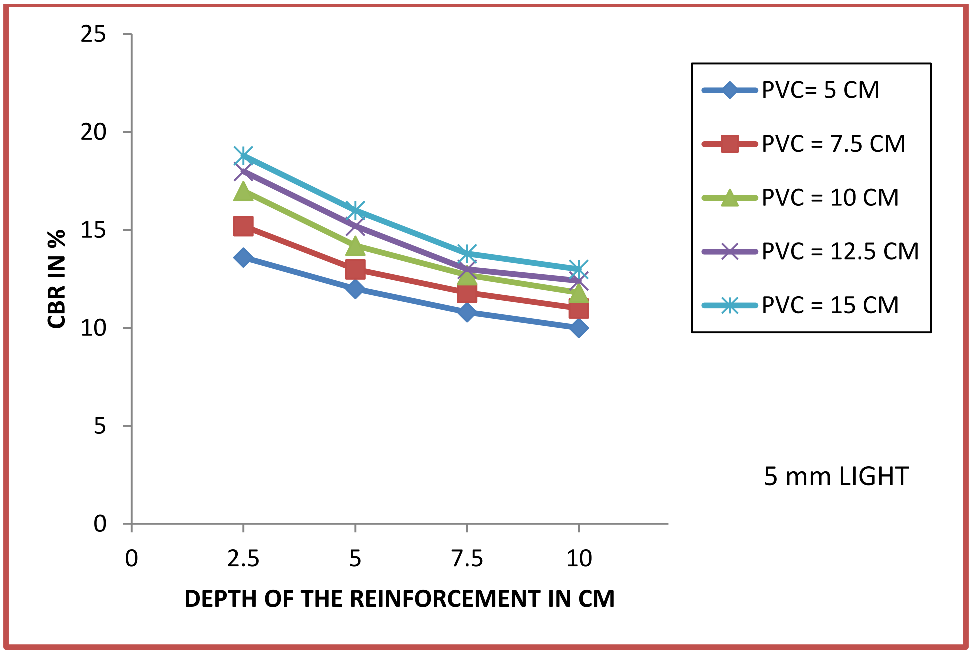

Fig 4.21 variation of CBR value with position of reinforcement (samples compacted at standard density)

For compacted at modified density:

2.5 mm penetration:

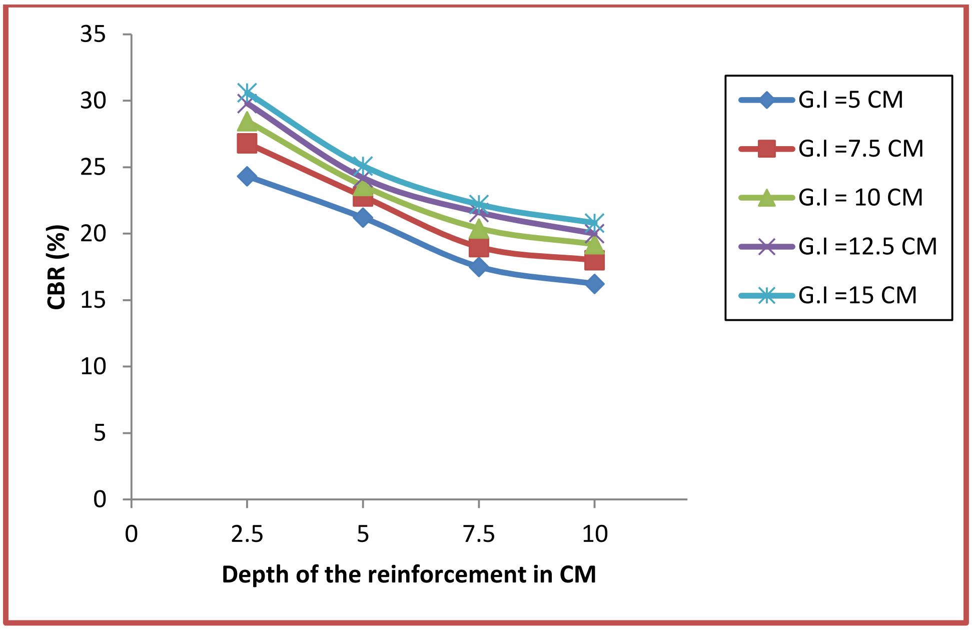

Fig 4.22 variation of CBR value with position of reinforcement (samples compacted at modified density)

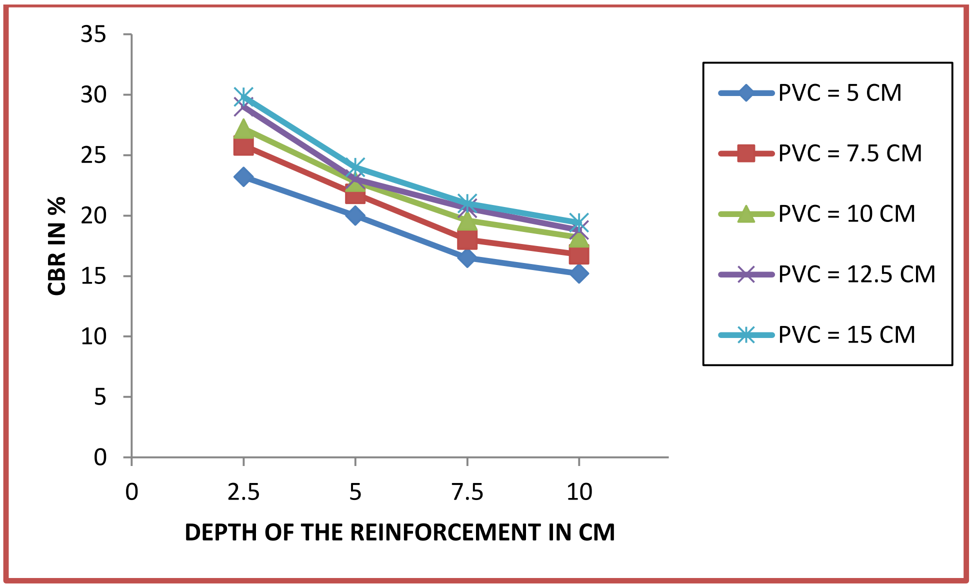

Fig 4.23 variation of CBR value with position of reinforcement (samples compacted at modified density)

At 5 mm penetration:

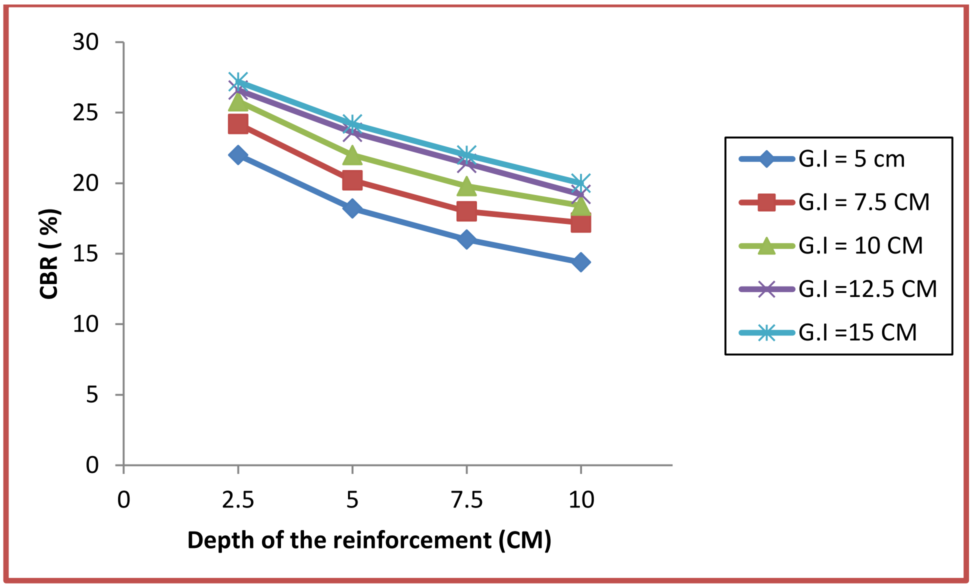

Fig 4.24 variation of CBR value with position of reinforcement (samples compacted at modified density)

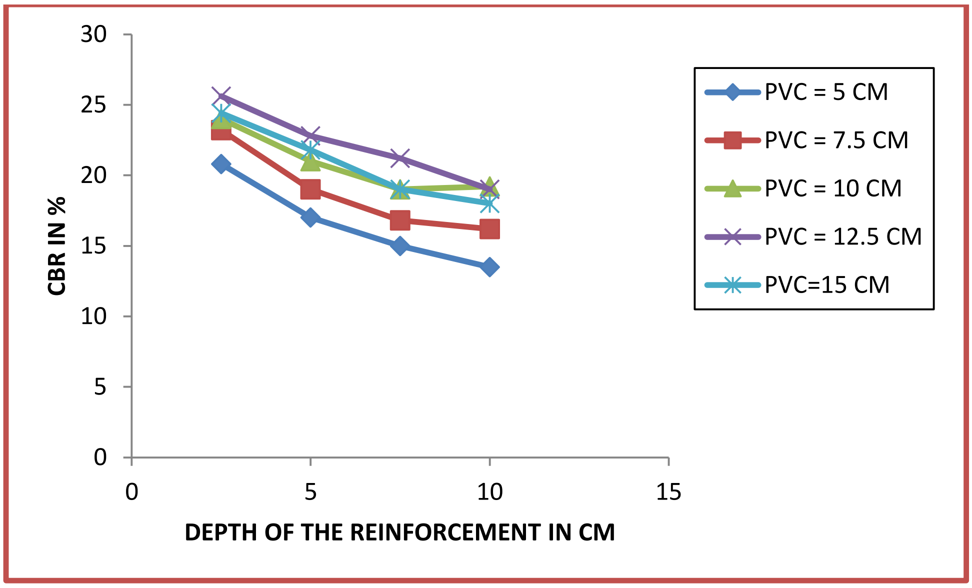

Fig 4.25 variation of CBR value with position of reinforcement (samples compacted at modified density)

TABLE 4.2 CBR VALUES OF REINFORCED POND ASH:

Position of rein forcement (depth)in cm	P.V.C.5 CM		P.V.C 7.5 CM		P.V.C 10 CM		P.V.C 12.5 CM		P.V.C. 15 CM	
	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM
2.5	16.2	13.6	17.2	15.2	18.4	17	19.3	18	20.4	18.8
5	13.4	12	14	13	14.8	14.2	15.8	15.2	17	16
7.5	12.4	10.8	12.8	11.8	13	12.7	14.8	13	15.4	13.8
10	11.2	10	12.2	11	12	11.8	13.8	12.4	14.4	13

Position of rein forcement (depth)in cm	G.I. 5 CM		G.I. 7.5 CM		G.I. 10 CM		G.I 12.5 CM		G.I 15 CM	
	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM
2.5	17.4	14.8	18.4	16.4	19.7	17.8	20.3	18.8	21.8	20
5	14.2	13	15	14	16.1	15.4	16.6	16.2	17.8	17
7.5	12.4	11.8	13.2	13	14.1	14.2	14.8	13.6	16.1	14.4
10	11.4	11	12	12.4	12.5	13.6	13.2	13	14	13.8

Position of rein forcement (depth)in cm	P.V.C.5 CM		P.V.C 7.5 CM		P.V.C 10 CM		P.V.C 12.5 CM		P.V.C. 15 CM	
	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM
2.5	23.2	20.8	25.8	23.2	27.2	24.8	29	25.6	29.8	24.4
5	20	17	21.8	19	22.8	21.2	23	22.8	24	21.8
7.5	16.5	15	18	16.8	19.6	20.6	20.6	21.2	21	21
10	15.2	13.5	16.8	16.2	18.2	19.2	18.8	18.4	19.4	17.8

Position of rein forcement (depth)in cm	G.I. 5 CM		G.I. 7.5 CM		G.I. 10 CM		G.I 12.5 CM		G.I 15 CM	
	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM	2.5 MM	5 MM
2.5	24.32	22	26.8	24.2	28.46	25.8	29.8	26.6	30.6	27.2
5	21.2	18.2	22.8	20.2	23.6	22	24.2	23.6	25.1	24.2
7.5	17.51	16	19	18	20.4	19.8	21.6	21.4	22.2	22
10	16.22	14.4	18	17.2	19.2	18.4	20	19.2	20.8	20

4.1.4 EFFECT OF REINFORCEMENT POSITION ON CBR VALUES (PVC & G.I):

Variation of CBR value with varying diameter of reinforcement for samples compacted at standard proctor density and modified proctor density reinforced with G.I and PVC reinforcements as shown in Figure. From the figure it is observed that the CBR values of pond ash increases with the increase in diameter of reinforcement located within two times the diameter of footing below top surface. It was found that the CBR value increases with increase in stiffness of reinforcement. In the present study, the diameter of footing varied from 5 cm to 15 cm with a constant increment of 2.5 cm and depth of reinforcement varied from 2.5 cm to 10 cm. Fig 4.26 to Fig 4.33 shows the variation of CBR value with size of reinforcement (compacted samples of both at standard density and modified density). It was observed that the CBR values of reinforced pond ash increases with increase in stiffness of reinforcement. At the same reinforcement depth, inclusion of GI material found to give more CBR value than PVC irrespective of the test condition.

At 2.5 mm penetration for standard:

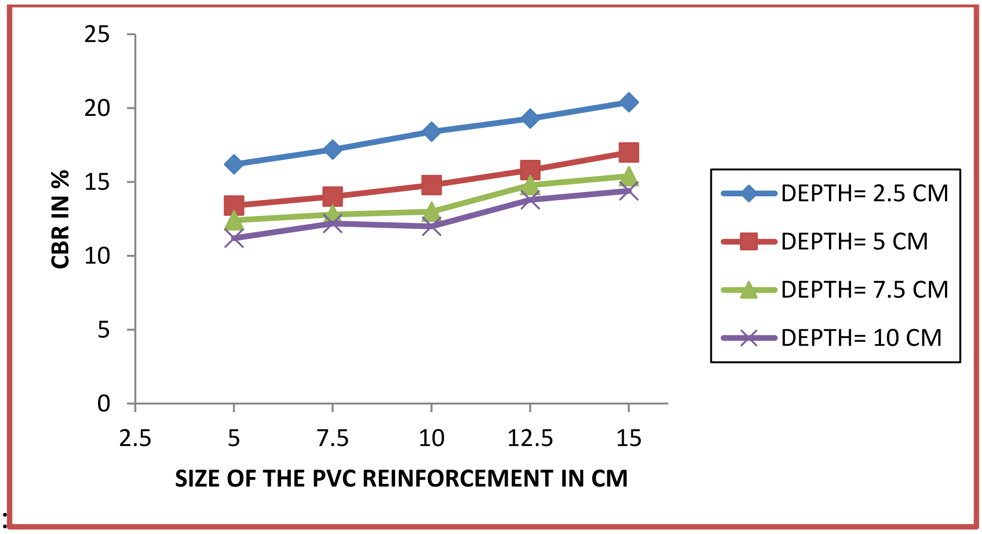

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Fig 4.26 variation of CBR value with position of reinforcement (samples compacted at standard density)

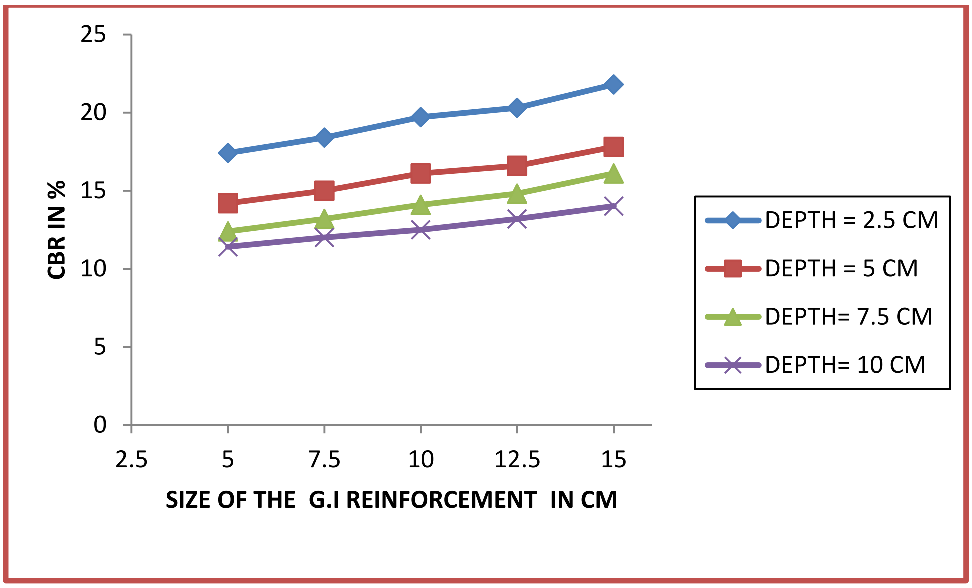

Fig 4.27 Variation of CBR value with position of reinforcement (samples compacted at standard density)

At 5 mm penetration:

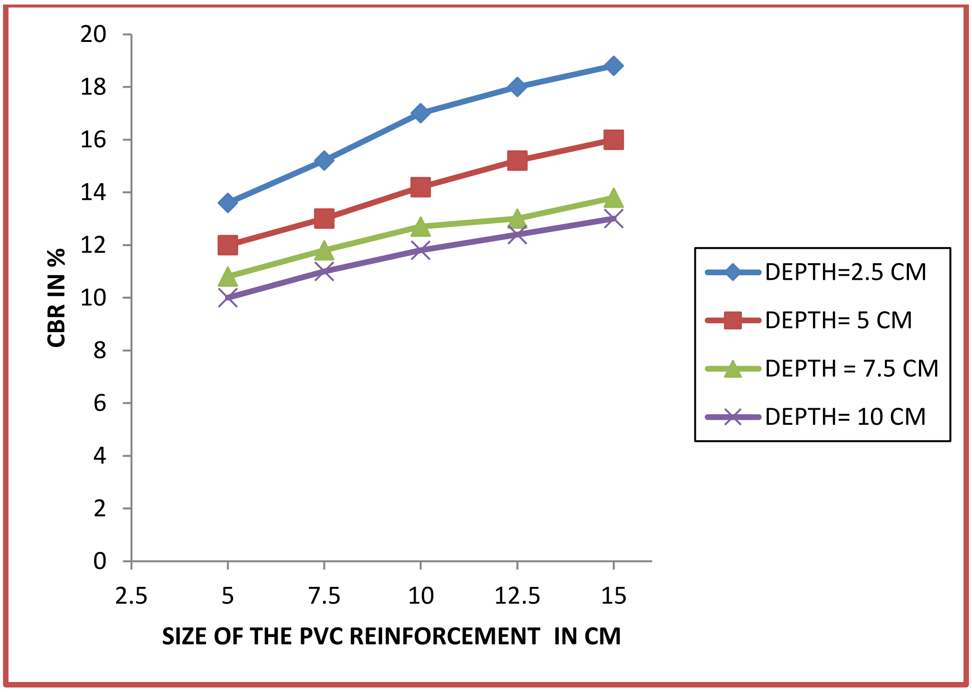

Fig 4.28 Variation of CBR value with position of reinforcement (samples compacted at standard density)

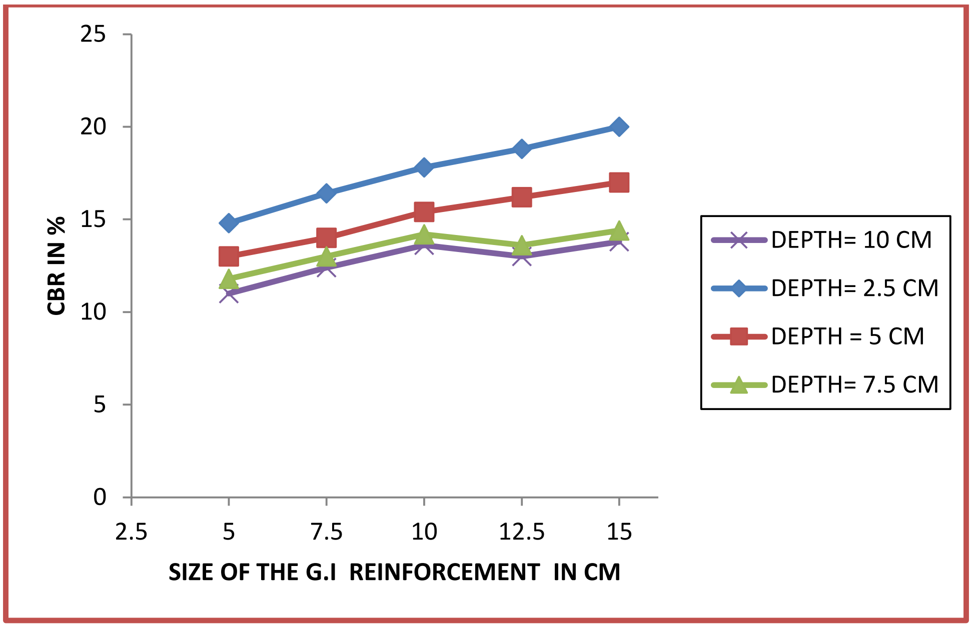

Fig 4.29 variation of CBR value with position of reinforcement (samples compacted at standard density)

At 2.5 mm penetration for modified:

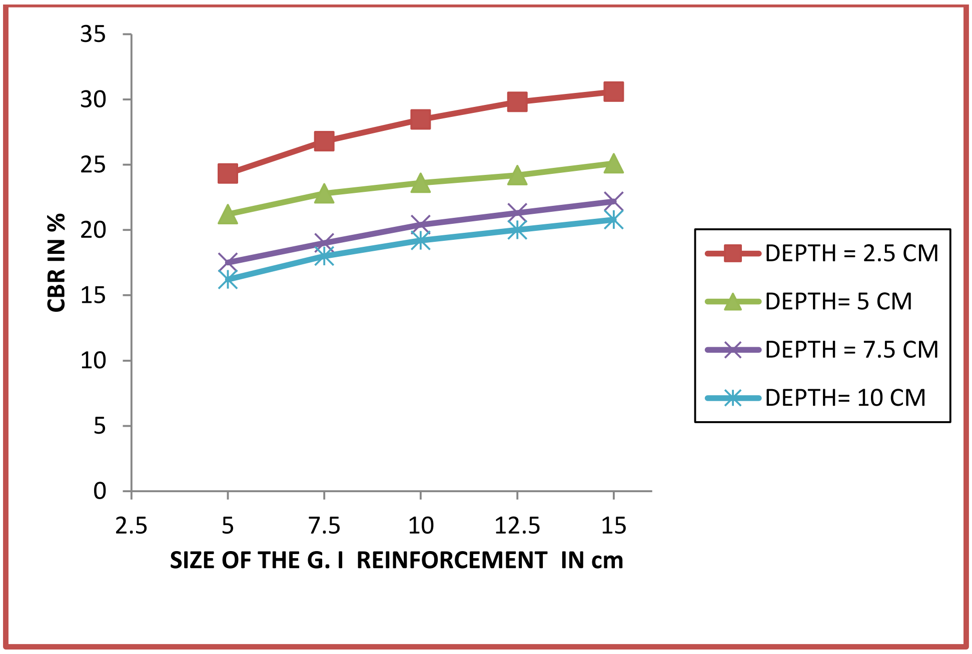

Fig 4.30 variation of CBR value with position of reinforcement (samples compacted at modified density)

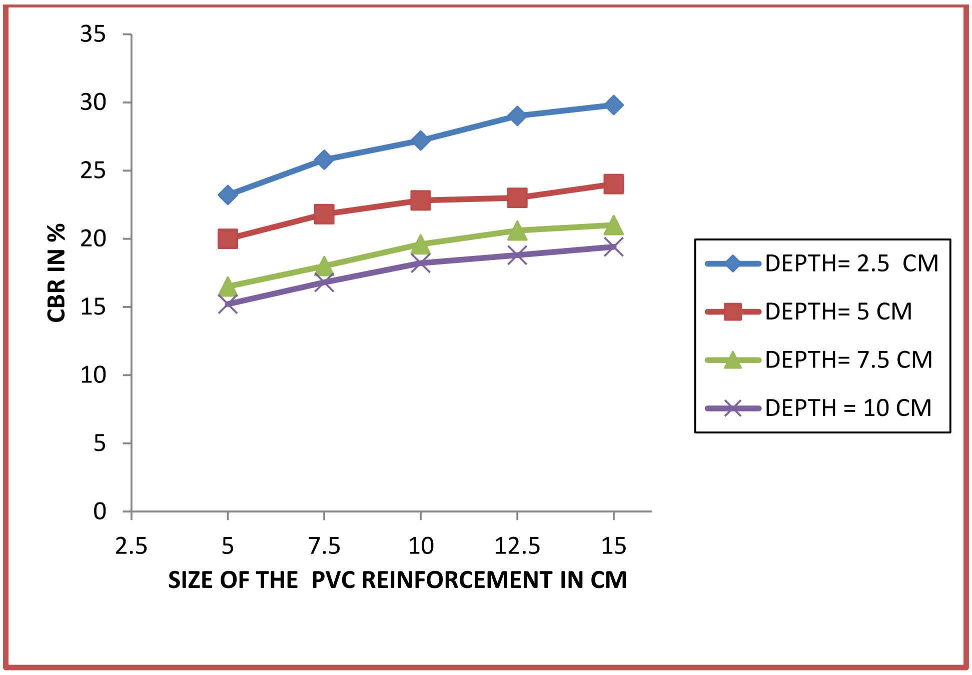

Fig 4.31 variation of CBR value with position of reinforcement (samples compacted at modified density)

At 5 mm penetration:

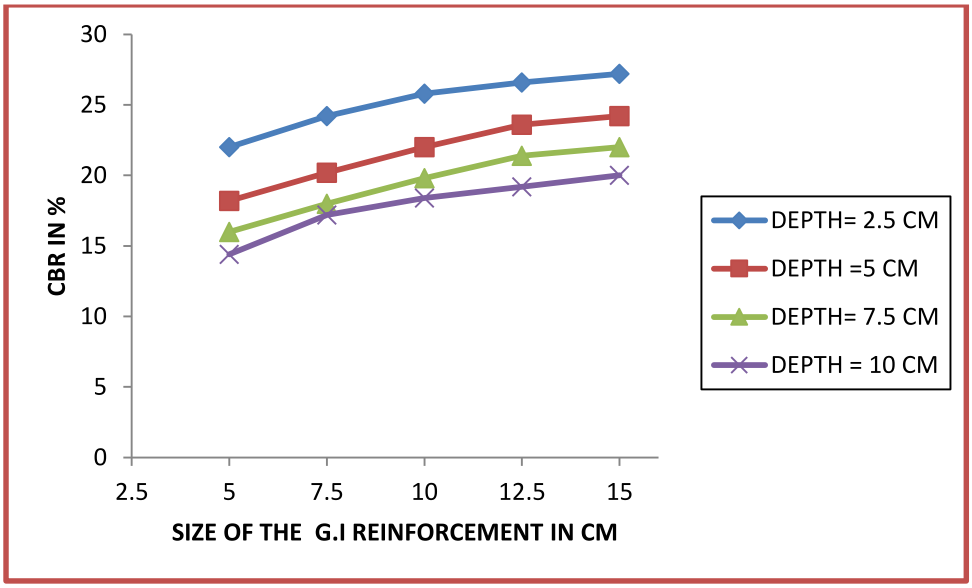

Fig 4.32 variation of CBR value with position of reinforcement (samples compacted at modified density)

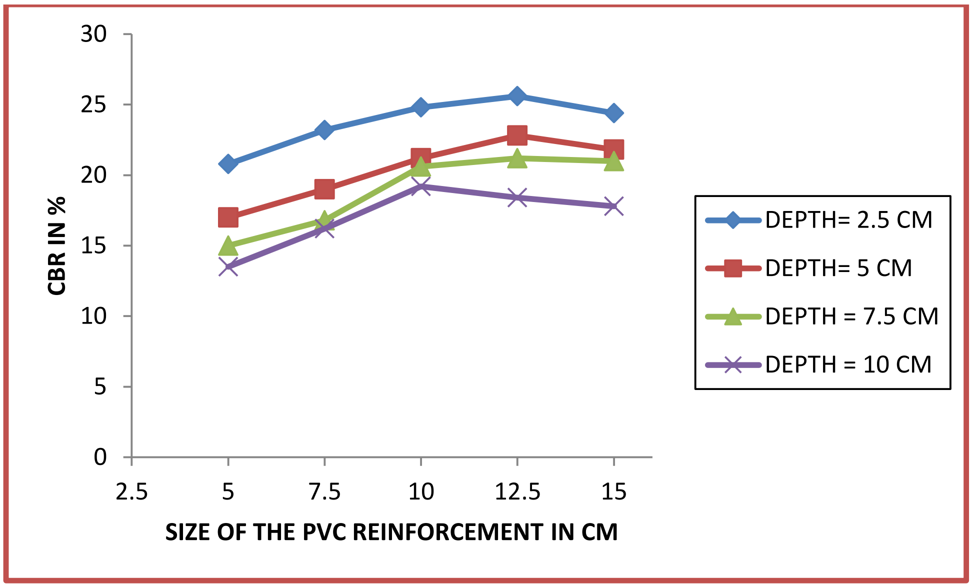

Fig 4.33 variation of CBR value with position of reinforcement (samples compacted at modified density)

4.2 LOAD DEFORMATION BEHAVIOR OF SAND WITH VARYING MOSITURE CONTENT AT DIFFFERENT RELATIVE DENSITIES:

In the present study, the load-deformation behavior of sand were studied by varying water content and density. Fig 4.34 to 4.36 shows the load deformation behavior of sand with varying moisture content at different relative densities. It is observed that, at the same water content, the peak load of samples compacted corresponding to lower RD found to be lower than the samples compacted corresponding to higher RD.

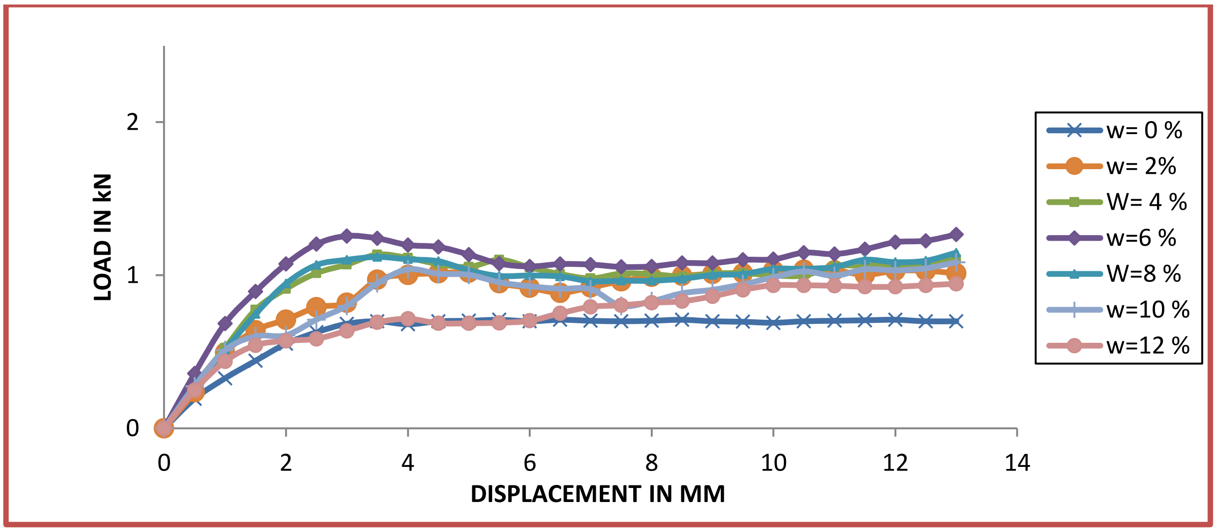

Fig 4.34 Load deformation behavior of sand with varying the moisture content (Relative density 30 %)

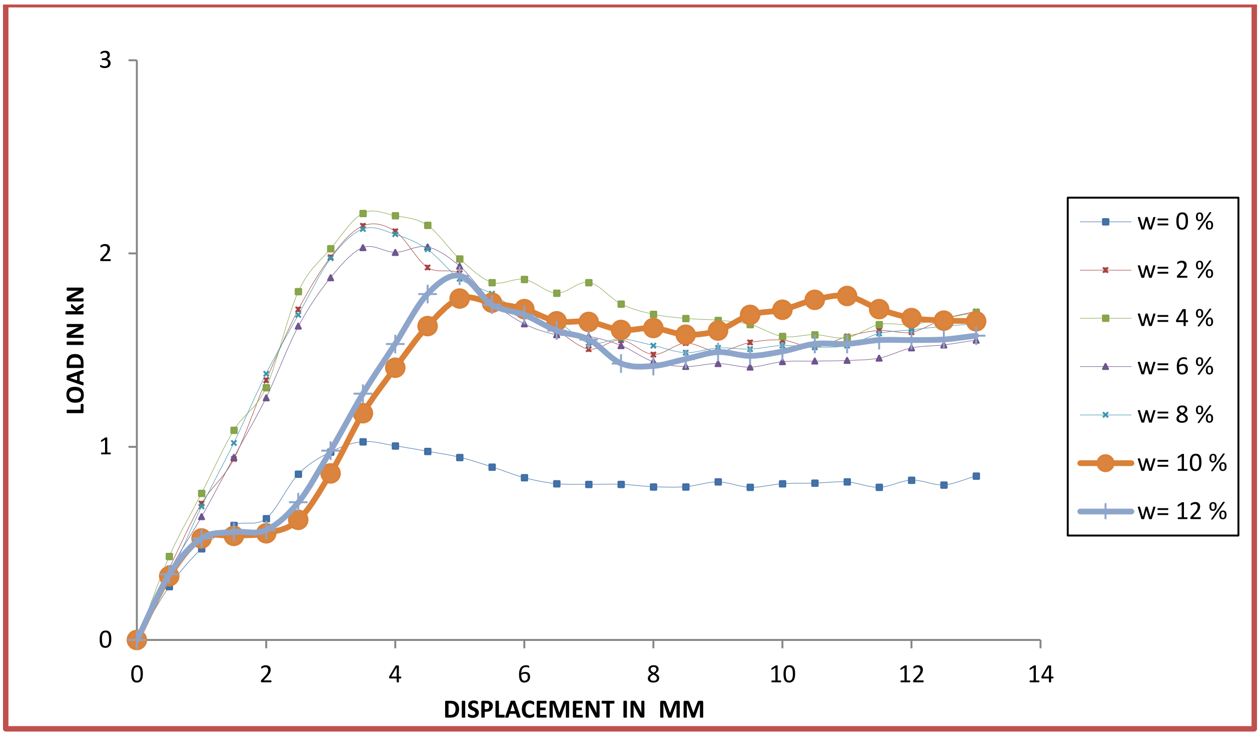

Fig 4.35 Load deformation behavior of sand with varying the moisture content (Relative density 60 %)

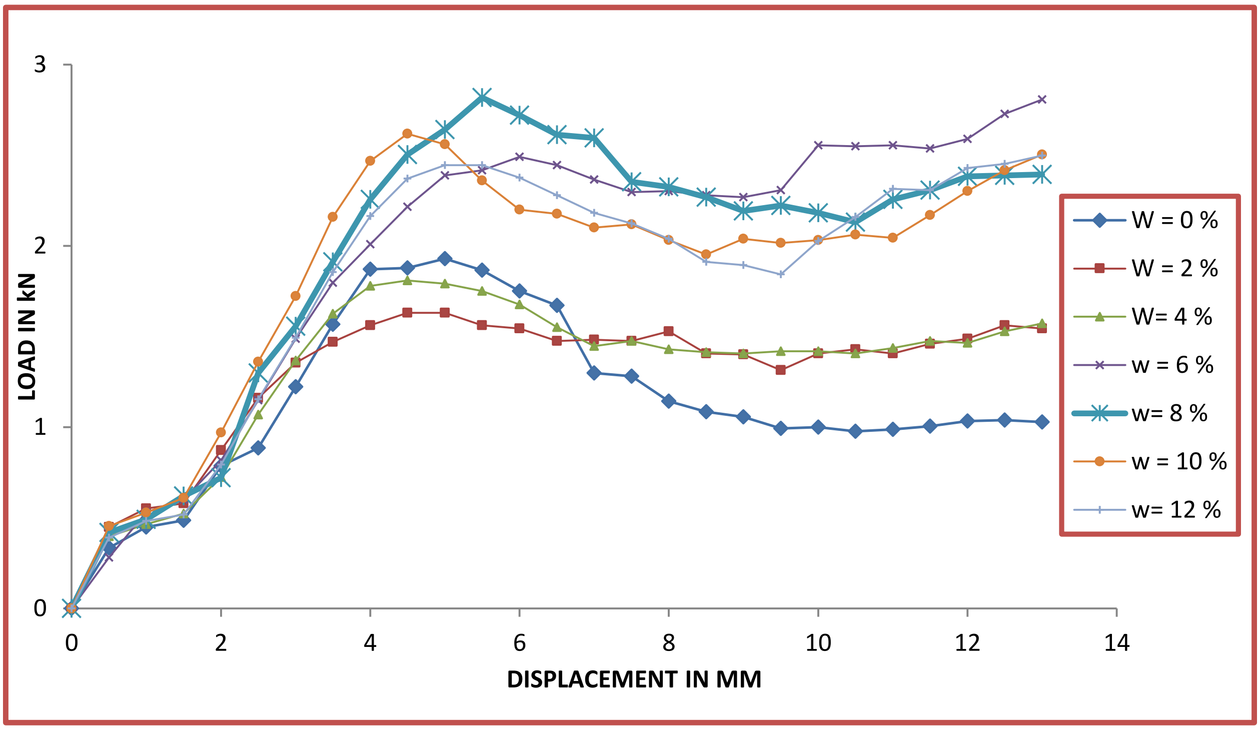

Fig 4.36 Load deformation behavior of sand with varying the moisture content (Relative density 90 %)

4.2.1 EFFECT OF MOISTURE CONTENT AND RELATIVE DENSITY ON CBR VALUE OF SAND:

The CBR test was conducted on sand by altering different relative density of (30%, 60%& 90%) on sand, and also the under different moisture conditions from 0-12%. Fig 4.37 and 4.38 shows the variation of CBR of sand with moisture content at different relative densities. It is observed that, at the same water content, the CBR value of samples compacted corresponding to lower RD found to be lower than the samples compacted corresponding to higher RD. There is notable variation in CBR value was observed with moisture content.

At 2.5 mm penetration:

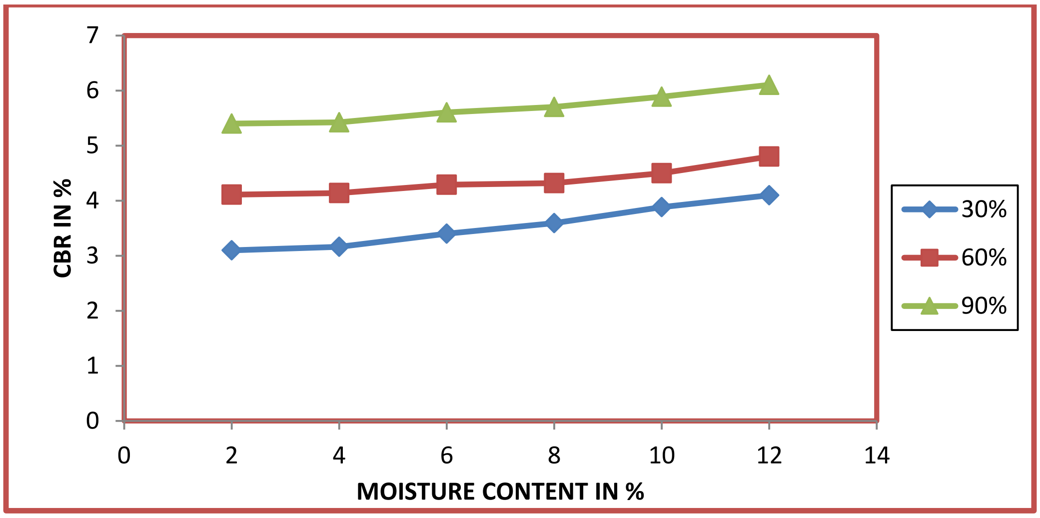

Fig 4.37 variation of CBR value with moisture content with different relative density

At 5 mm penetration:

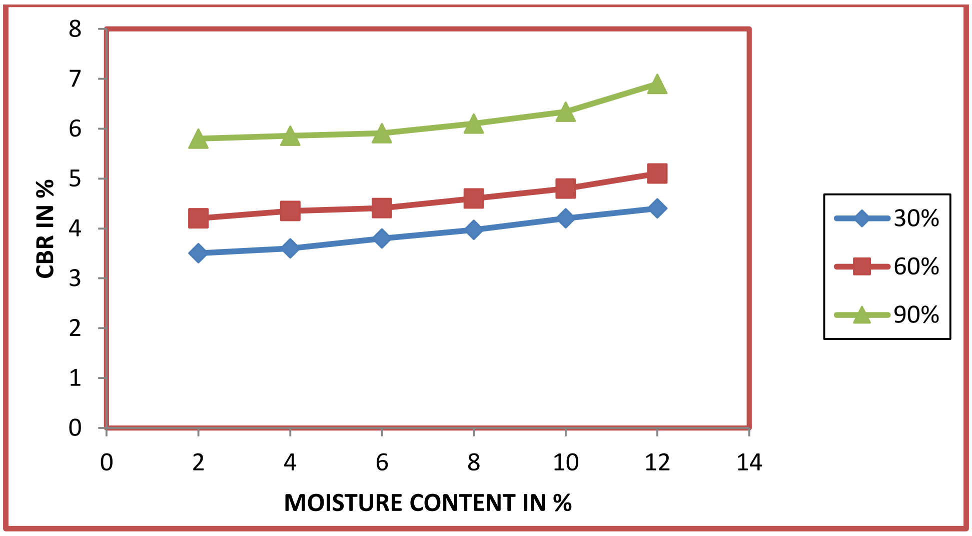

Fig 4.38 variation of CBR value with moisture content with different relative density

4.3 Effect of overlain sand thickness on CBR value of compacted pond ash:

CBR values of pond ash compacted to standard proctor density and over lain by a layer of sand of varies thickness with relative density of 90 % were calculated. The thickness of the over lain sand layer were varied from 0 to 20 cm. The variation of CBR value with the thick ness of over lain sand layer is presented in figure 4.39. It is found that, as the thickness of sand layer increases the CBR value increases steadily and attains a least value equal to the CBR value of sand at 90 % relative density. This shows that when the thickness of overlain sand layer is more than 4 times the diameter of the loaded area the effective of under lain compressive layer the soil is not felt. As the compacted sand layer dose not decrease its CBR value were as the pond ash beds loses its strength subsentially. A layer of sand of 4 times diameter of the loaded area can be used to improve its load carrying capacity.

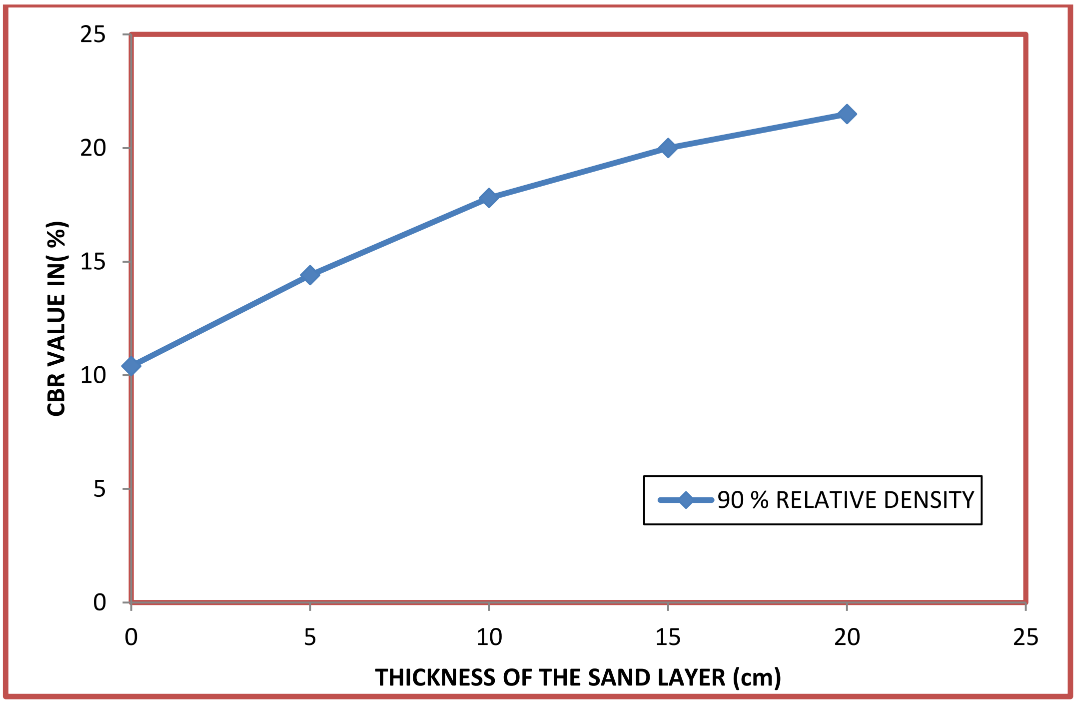

Fig 4.39 variation of CBR value of compacted pond ash with overlain sand thickness (samples compacted at standard density at relative density 90 %)

CHAPTER 5

CONCLUSION:

Based on experimental results are following conclusion are derived..

1. Pond ash compacted at modified proctor density gives approximately 2 times higher CBR value than sample compacted at standard proctor density
2. For a given location of reinforcement as the size of reinforcement increases the CBR value increases.
3. As the CBR value decreases as the depth of reinforcement increases. When the depth of reinforcement is 2 times the diameter of the plunger, practically there is no improvement in the bearing capacity of reinforced pond ash over unreinforced one.
4. Addition of small amount of water to sand increases the CBR value .this may be due to the development of pseudo- cohesion in partially saturated sand .with increasing a relative density of sand the CBR value increases.
5. The CBR value of compacted pond ash can be improved by putting a layer of dense sand over it a sand thickness of 4 times the diameter of the plunger gives a CBR value which is equal that of a dense sand only. Hence it is concluded that if the thickness of overlain dense sand layer is 4 times the diameter of the loaded area the effect of under lain, soft compressible layer is not felt.

SCOPE OF THE PRESENT STUDY:

Other reinforcement materials like fiber, geogrid can be used to study the effectiveness of improvement.

Other cementing materials like cement, lime be used alone or in combination to study the improvement in bearing capacity.

Interfacial shear resistance of reinforcement to be studied.

Correlation can be formulated by considering reinforcement property, geometry property of footing and property of soil domain.

CHAPTER 7
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REFERENCES:

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